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UPGRADING BASEMENTS FOR COMBINED NUCLEAR WEAPONS EFFECTS: PREDESIGNED EXPEDIENT OPTIONS II

Final Technical Report

July 1980

Contract No. DCPA01-77-C-0227
FEMA Work Unit No. 1155C

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H. L. MURPHY ASSOCIATES

Box 1727
San Mateo, California 94401
(415) 348-2180

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Page	Comment
A1-22	In 4th line from bottom: Correct spelling to "static".
A2-9	In footnote: Insert parenthesis to change to "... table) is ...".
A2-20.1	In 4th column, heading: Correct spelling to "Length".
B2-8	In Figure B2-1A: Correct arrow for "Cap Block" to run to block that is on top of "Post".
D1-13	In definition of "ksi": Change "kip" (within parentheses) to "ksi".

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20 ABSTRACT

By: H. L. Murphy*

This report covers the results of the latest phase of a 3-phase project, with the overall objective of developing a set of expedient and engineered techniques, for upgrading the air blast and related effects resistance potential of basements in existing buildings. The purpose of upgrading such basement spaces is to provide shelter when needed by persons in: host areas, where the bulk of the population is expected to be during an attack, that are located at and beyond the 2-psi air blast range, using selected target aiming points and Mt-range bursts; and, risk areas, where shelter is needed that is within 15-minutes travel time of each key worker's place of work and provides potential shelter for 30- to 50-psi air blast ranges, in terms of peak free field overpressure.

Chapters of this report's main text are devoted to discussions of: background; general principles applicable to upgrading basements; closures for all basement shelter openings/apertures, in terms of principles for providing them; needs to be met in strengthening the structure over shelter candidate basements; some techniques and materials that can be used for such structure strengthening; and, shelters for key workers. In general, the main text of this report is intended for the artisan, the appendices having the more extensive, technical data and discussions.

The titles of the appendices are: Blast-Resistant Design/Analysis General Approach; Plywood Stressed-Skin Panels (Two-Sided Only) as Closures - Design and Fabrication; Plywood Stressed-Skin Panels (Two-Sided) as Beam-Columns; Plywood Use for Closures - Design; Wood Beam and Column Design - Simply Supported; Home Basements Upgrading in Host Areas; Blast-Resistant Design/Analysis of Steel Members; and, Structural Steel Local Availability and Use for Blast Shelter Upgrading.

The suggestions, guidance and technical help of M. A. Pachuta, G. N. Sisson, and D. W. Bensen, FEMA, are gratefully acknowledged, as are the contributions of former colleagues C. K. Wiehle, E. E. Pickering and J. E. Beck.#

* H. L. Murphy Associates, Box 1727, San Mateo, CA 94401 (415-348-2180)
# James E. Beck and Associates, 4216 Los Palos Av., Palo Alto, CA 94306

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Summary

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By: H. L. Murphy
Consulting Civil Engineer*

For: Federal Emergency Management Agency
Washington, D.C. 20472

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(415) 326-6200 • Cable: SRI INTL MPK • TWX: 910-373-1246

*H. L. Murphy Associates, Box 1727, San Mateo, CA 94401; 415-348-2180

SUMMARY

This report covers the results of the latest phase of a 3-phase project, with the overall objective of developing a set of expedient and engineered techniques, for upgrading the air blast and related effects resistance potential of basements in existing buildings. It is recommended that any serious user of this report have copies of the reports covering the first two phases at hand. The purpose of upgrading such basement spaces is to provide shelter when needed by persons in host or risk areas or elsewhere. As stated in the second phase report, some work results were reported there for which the work was actually performed under this third phase's funding.

The first two phases of work were performed under a policy guidance that called for exploiting the inherent ultimate blast resistance of the slab over the basement selected for upgrading consideration, that is, upgrading work that would strengthen the remainder of the floor support system to a level equaling the inherent strength of the floor slab. The hoped for result was shelter adequate for, say, 8 to 15 psi blast free field overpressure. Exceptions were made, of course, such as the case of a large basement cut in half by a long-way interior corridor, and covered on each half with a rather long span, one-way slab running between corridor walls and outer walls; the basement cover slab's potential blast resistance could be upgraded considerably by simply running added support walls parallel to the interior corridor and supporting each existing one-way slab at its midspan.

In this (third) phase of the project work, shelter guidance for selection of candidate basements for upgrading has been re-oriented to meet the CRP (Crisis Relocation Plans). The CRP requires shelter in two kinds of specific geographic areas, which are located by study of selected attack areas in the United States and relative air blast ranges from the targets:

1. Host areas, where the bulk of the population is expected to be during an attack, that are located at and beyond the 2-psi air blast range, using the selected target aiming points and Mt-range bursts. Circumstances have required, in certain cases, a policy exception to provide shelter at and beyond the 3 psi range; the appendix on Home Basement Upgrading for Host Areas is specifically aimed at meeting this shelter need.

2. Risk areas, where shelter is needed that: (1) is within 15-minutes travel time of each key worker's place of work; and (2) provides potential shelter suitable for 30 to 50 psi air blast peak free field overpressure. Such shelter is discussed in Chapter 7.

The remaining chapters of this report's main text are devoted to a more complete background discussion; a discussion of general principles applicable to upgrading basements; a discussion of closures for all

basement shelter openings/apertures in terms of principles for providing them; a discussion of the needs to be met in strengthening the structure over shelter candidate basements; a discussion of some techniques and materials that can be used for such structure strengthening; and, a discussion of shelters for key workers as just mentioned. In general, the main text of this report is intended to be understandable to the non-engineer/architect and to provide help, for such a person as an artisan semiskilled in carpentry, in dipping briefly into several of the technical appendices to use them without having to either study or understand completely each entire appendix.

The titles of the appendices are: A. *Blast-Resistant Design/Analysis General Approach*; A1. *Plywood Stressed-Skin Panels (Two-Sided Only) as Closures - Design and Fabrication*; A2. Plywood Stressed-Skin Panels (Two-Sided) as Beam-Columns; A3. Plywood Use for Closures - Design; B1. Wood Beam and Column Design - Simply Supported; B2. Home Basements Upgrading in Host Areas; D1. Blast-Resistant Design/Analysis of Steel Members; and, E1. Structural Steel Local Availability and Use for Blast Shelter Upgrading.

Acknowledgments

Through suggestions and guidance, the technical help of M. A. Pachuta, G. N. Sisson, and D. W. Bensen, U. S. Federal Emergency Management Agency, was freely given and is gratefully acknowledged. Similarly acknowledged are: the work of a colleague, J. E. Beck,[1] in assisting specifically in preparing Appendix A2, as well as contributing pieces, plus advice in technical discussions, used in many places in this report; considerable work earlier in this project phase by another colleague, C. K. Wiehle; and the work of E. E. Pickering[2] in preparing Appendix E1.

[1] Formerly at SRI International and currently the principal of James E. Beck and Associates, Palo Alto, California.

[2] Consulting Civil Engineer, Menlo Park, California.

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UPGRADING BASEMENTS FOR COMBINED NUCLEAR WEAPONS EFFECTS: PREDESIGNED EXPEDIENT OPTIONS II

Final Technical Report

July 1980

By: H. L. Murphy
Consulting Civil Engineer*

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*H. L. Murphy Associates, Box 1727, San Mateo, CA 94401; 415-348-2180

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FIGURE

Chapter 1

INTRODUCTION

This report covers the results of the latest phase of a 3-phase project, with the overall objective of developing a set of expedient and engineered techniques, for upgrading the air blast and related effects resistance potential of basements in existing buildings. Reports covering the first two phases are References [1 and 2].[1] It is recommended that any serious user of this report have copies of both references at hand.[2] The purpose of upgrading such basement spaces is to provide shelter when needed by persons in host or risk areas or elsewhere. As stated in the footnote on page 1 of the second phase report [Ref. 2], some work results were reported there for which the work was actually performed under this third phase's funding.[3]

The first two phases of work were performed under a policy guidance that called for exploiting the inherent ultimate blast resistance of the slab over the basement selected for upgrading consideration, that is, upgrading work that would strengthen the remainder of the floor support system to a level equaling the inherent strength of the floor slab. The hoped for result was shelter adequate for, say, 8 to 15 psi blast free field overpressure. Exceptions were made, of course, such as the case of a large basement cut in half by a long-way interior corridor, and covered on each half with a rather long span, one-way slab running between corridor walls and outer walls; the basement cover slab's potential blast resistance could be upgraded considerably by simply running added support walls parallel to the interior corridor and supporting each existing one-way slab at its midspan.

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- Host areas, where the bulk of the population is expected to be during an attack, that are located at and beyond the 2-psi air blast range, using the selected target aiming points and Mt-range bursts. Circumstances have required, in certain cases, a policy exception to provide shelter at and beyond the 3 psi

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1 Brackets are used herein to indicated sources in the References list at the end of this report (main text).

2 Available for purchase from NTIS, Springfield, Virginia 22151.

3 For example, original Appendices A1 and A3 herein.

range, meaning that Appendix B2, Home Basement Upgrading for Host Areas, is specifically aimed at meeting this need for shelter against 2-3 psi air blast peak free field overpressure.

- Risk areas, where shelter is needed that: (1) is within 15-minutes travel time of each key worker's place of work, and (2) provides potential shelter suitable for 30 to 50 psi air blast peak free field overpressure. Such shelter is discussed further in Chapter 7.

The remaining chapters of this report's main text are devoted to a more complete background discussion; a discussion of general principles applicable to upgrading basements; a discussion of closures for all basement shelter openings/apertures in terms of principles for providing them; a discussion of the needs to be met in strengthening the structure over shelter candidate basements; a discussion of some techniques and materials that can be used for such structure strengthening; and, a discussion of shelters for key workers as just mentioned. In general, the main text of this report is intended to be understandable to the non-engineer/architect and to provide help, for such a person as an artisan semiskilled in carpentry, in dipping briefly into several of the technical appendices to use them without having to either study or understand completely each entire appendix.

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[4] Formerly at SRI International and currently the principal of James E. Beck and Associates, Palo Alto, California.

[5] Consulting Civil Engineer, Menlo Park, California.

Chapter 2

BACKGROUND

The Introduction section of the second phase report [2] has been updated and is used below.

Because of their inherent shelter potential, basements of substantial buildings having a concrete slab first floor (supported by either steel or reinforced concrete beam-girder-column systems) are the natural choice for relatively high degrees of protection. The first floor (floor over basement) concrete slab ordinarily has a rather high degree of air blast (collapse) resistance, because for normal use it must withstand individual point loads, as well as general area loads. The supporting beams and girders become progressively weaker, in that order (in ultimate or collapse strength), however, as the tributary areas served increase in size and thus the effect of normal use point loads decreases for these heavier supporting members. Columns may or may not be relatively weaker depending on the height of the building. It is frequently found that the first floor slab itself will resist about 10 psi peak "side-on" air blast pressure or more, but that this potential is degraded rather seriously by weaker beams and girders, and sometimes columns. In addition most basements have exterior openings, and all have interior openings, permitting air blast to enter (assuming that the air blast wave passes through or destroys the building above the basement). Thus, the general principle to be followed is to exploit the relatively high floor slab resistance, through closing openings as well as applying strengthening measures to the other portions of the first floor system and possibly to the basement column system. If the first floor slab does not have an acceptable level of inherent blast resistance, upgrading the basement may still be well worth considering but should be balanced against the cost (in materials, elapsed time and manpower used) of alternate schemes for shelter (e.g., expedient/field shelters; constructed R/C (reinforced concrete) and/or corrugated metal ducts (see Chapter 7); mines, etc.).

The measures available for upgrading existing basement space for shelter purposes may be categorized as either "expedient" or "engineered." Expedient measures are those that can be accomplished in a relatively short period of time (say two to three days) during a crisis build-up period by building occupants using readily available materials. Expedient measures may be pre-engineered with resulting designs distributed in advance in "how-to-do-it" drawings and instructions. Engineered measures also require longer periods of time and the services of professional engineers for evaluation and design, perhaps tailored to a specific building or a specific type of building.

Upgrading measures considered include prevention of air blast entry into the shelter space, reduction of air blast loading on exposed areas, strengthening of floor system structural members, provision of debris protection, provision of "last resort" shelter in case of floor system collapse, and other protective measures. Both closed and open shelter situations were considered as were post-attack considerations.

For the expedient case, the most common vulnerability problems were examined and principles of protection are given. Specific building features requiring protection are illustrated and suitable methods of protection and materials are presented. The degree of protection afforded by the various methods and materials are given. Suggested local sources of materials and required tools are also given. The expedient section is prepared in "how-to-do-it" illustrative manner so as to permit ready application by non-engineer building occupants and other untrained personnel.

For the engineered case, the air blast resistance characteristics of suitable basements in existing buildings are described along with upgrading principles and techniques. Methods of evaluating individual buildings for basic first floor system air blast resistance are discussed. Upgrading design guidance for various building features is given. Specific detailed evaluation and design procedures for the more complex upgrading problems are given in appendices. Several examples of existing buildings are also given, with basement upgrading measures applied.

It is intended that this report, together with the first- and second phase reports [1,2] serve as a basic reference and guidance for civil defense planners, building owners, occupants charged with upgrading shelter space for themselves, engineering enterprises, and others concerned with the air blast upgrading of existing buildings before or during a strategic population relocation, or other civil defense shelter program. The information contained herein will also be useful for expedient upgrading, on an opportunity basis, of buildings used for temporary shelter in the population relocation or "host" areas.

Many of the matters mentioned above were covered in the first-phase report [1] for which an overview is provided by including that report's Contents, Table and Figures lists; see Table 1. Figures 1 through 11 of the earlier report [1] provide schematics or concepts for closures and structural strengthening; the later work reported in Reference [2] and hereinafter includes engineered/predesigned data for use in closures and in structure strengthening.

For technical readers such as civil engineers and architects interested in strengthening of basements for combined nuclear weapons effects shelter, recommended reading includes the Appendices herein plus References [1 and 2] for existing basements, and References [3 and 4] for basements under design or planning.

Table 1 CONTENTS, TABLE AND FIGURES LISTS FROM REFERENCE 1

CONTENTS

Table 1 (continued)

Table 1 (concluded)

TABLE

FIGURES

Open Versus Closed Shelters

Some vulnerability problems of open and closed shelters are discussed starting on page 6 of Reference [1], as indicated in its Contents list reproduced herein in Table 1. Certain matters, however, are worth brief mention here.

Open shelter requires that all materiel in the shelter be fastened with sufficient strength to resist the anticipated entering blast wave, or else such items of materiel may well become missles causing injury or death. An alternative is to surround the materiel items that pose potential missle hazards with a blast resistant barrier, so that the blast wave does not strike the items. Dealing with such potential missle hazards can be expensive (in terms of materials, elapsed time, and manpower), varying with the type of structure: for example, a parking garage might be provided with such missle hazard reduction on a cost-effective basis, whereas a potential shelter space with many pieces of equipment of varying degrees of fixity could well dictate against considering open shelter.

Closed shelters can be provided with aperture closures (see Chapter 4) and, within the range of air blast peak free field overpressures considered in this study, that is up to 50 psi, materiel will generally not require fastening in place; exceptions would be made in the case of overhead items by insuring that the fastenings are adequate to take any shock or vibration hitting the overhead structural members.

For host areas, where shelter against air blast up to 2 or 3 psi (see Chapter 1) are contemplated, work in anchoring materiel items might be expected to be modest in cost.

Subject to these brief comments, this report generally contemplates the use of closed shelters, unless specifically mentioned otherwise. A parking garage, for example, could well be the exception because of the problems of providing closures for the vehicle entrances in contrast to the cost of anchoring down or removing materiel items.

Appendices

The appendices of this report include several that are new to the work and others that are extracts, revisions, or provided totally by reference to a previously published appendix. Because of the manner of their development, the appendices are not numbered in the order of their use (as perhaps they should be).

Chapter 3

UPGRADING EXISTING BASEMENTS - GENERAL

This report is concerned with upgrading work for nuclear effects shelter in existing building basements, that are fully or partially below ground level and that can be bermed with soil up to the level of the first floor slab. If floor-level berming simply cannot be accomplished for the entire basement, the lessened radiation protection, as well as the need for increased blast protection of exposed wall portions (peak reflected pressure from a blast wave hitting a wall head-on is approximately three times the peak blast free field overpressure), must be considered. Generally, use of only part of the basement for closed shelter is more costly (in terms of available resources) than use of the full basement, because of the need for blast resistant interior walls around the partial basement shelter.

Radiation protection upgrading, whether for fallout or initial nuclear and thermal, usually requires simply additional mass in or on the floor over the shelter, plus of course the berming mentioned above. This can be bagged material or loose soil (easily placed on the floor slab), in sufficient amount for the required level of protection generally stated in full policy statements only briefly mentioned earlier. Use of part of the (ultimate/collapse) strength of the overhead slab will, of course, reduce the strength available for blast resistance. Calculations involved in such portioning of the overhead floor strength are illustrated by examples in Appendix B2 on upgrading home basements, but would be similar for a floor over higher blast level shelters.

Blast resistance needs to be considered in upgrading can generally be described under the following three categories:

(1) Peak air blast free field overpressures up to 2 or 3 psi: As briefly mentioned earlier, this level of blast resistance applies to host areas used under the CRPs (Crisis Relocation Plans). In home basements, one can assume that the interior pressures will have a long enough rise time to reach the peak pressure, to allow handling the blast loading as a statically applied but very short duration load - stated another way, to consider the blast load as an impact load only in terms of duration, not in terms of an instantaneously applied load. Further, one should consider the ultimate/collapse strength blast resistance of the floor over the basement, not with the usual design factors of safety. These matters are discussed in more detail in Appendices A and B2.

(2) Blast resistance equalling a free field overpressure that will fully tax the inherent blast strength of existing slabs over the basement shelter to be upgraded: Although this level of blast resistance is not current but earlier policy, as discussed above, this level may represent the most that can be done for many basements considering the

available time and material sources, and/or may be the most cost-effective shelter when considered against constructing other shelter, such as expedient shelter schemes [5,6], buried conduits/large pipes, etc. This blast resistance level should work out to roughly 10 psi for a sizable number of existing basements or, say, 6-15 psi as a range.

(3) Blast resistance of 30-50 psi peak free field overpressure, the level needed for risk areas as shelter for key workers, again under CRPs (Crisis Relocation Plans): Work to date has indicated a low probability of getting this protection level of shelter in existing buildings, unless a truly high level of manpower and material resources is spent and, even then, serious questions need to be first answered by tests concerning the blast resistance that can be expected from existing exterior basement walls. This shelter level and related problems are discussed further in Chapter 7.

Another general or common technical matter that should be mentioned at this point is that of bearing between existing members and upgraded members, and their loads and supports. Bearing problems are discussed frequently in the following portions of this report, in terms of specific building materials, but two points are worth making at this juncture: first, if two different materials, or two different strength levels of the same material (e.g., wood), must bear on each other, the required common bearing area will, of course, be determined by that required for the weaker material; and, second, bearing damage, even destruction, may be acceptable in upgrading design, because of the 1-shot/1-time loading assumed for nuclear shelter design and the energy absorbed in damaging components such as the wood cribbing members or the wood wedges under columns.

For the last general upgrading suggestion, the reader/user of this report is encouraged to review Reference [1] for its eleven upgrading schemes shown in pages 15-33, specifically Figures 1 through 11 of that report. While these figures are not to scale and are thus called upgrading schemes herein, the following chapters and appendices will give the reader/user the means to "size" the members, whether beams, columns, or beam-columns, at least in some of the materials contemplated for use.

Chapter 4

CLOSURES

Closure is used herein to describe the upgrading covering put over all apertures or openings in the existing basements selected as potential shelter, whether the opening is for a window, door, ventilation, utility, or other, whether small or large (for a 2-in. pipe or a double door). In planning for each closure, shelter ventilation should be continually kept in mind, and the closure be planned for its ability to remain open as long as possible if such is needed for ventilation; for example, a vertical opening might use a steel plate, guillotine-type closure, that need only be tripped to fall into place, perhaps the quickest operating of any closure type.

Vertical closures should be avoided to the maximum extent possible, simply because they usually involve dealing with the blast load's peak reflected pressure rather than its side-on pressure on the surface of the ground, the difference being an approximate three times increase in peak pressure level, if one has to deal with reflected pressure created by the blast wave striking a vertical wall, or going down a window-well and striking an end wall. This ratio of reflected to side-on pressure varies considerably with conditions, but to handle the simple situation of a blast wave travelling along the ground and striking a vertical surface head-on, with a reasonable distance to travel to clear around the vertical surface, the ratio is approximately 3 to 1 (at least in the range of free field overpressures of interest herein); more precise ratios may be obtained by referring to the scale shown on page B1-26 of Appendix B1. Perhaps the commonest example that might be used would be that of a window-well, serving the basement of a building from large to single family residence in size; a closure of the guillotine-type might well be mounted on the outside of the exposed basement wall to drop down over the well opening into the building, but such planning would require this closure to bear the multiple-reflected pressure caused by the blast wave entering the window-well. A simpler and far better solution might be to put a closure over the top of the window-well, preferably at ground surface, and cover it with berming material (soil, sand, whatever); the closure might be a plywood panel or a steel plate, among others. In later chapters, where the sizing of the closure, beam, column, etc., is described, the anticipated peak applied blast loading is used, leaving it to the reader/user to make this peak applied blast loading the one that is felt by the closure or structural member, whether it be the peak reflected (head-on) blast loading or a peak side-on blast loading.

An easily overlooked item in the matter of closures is that of the need for examining the blast resistance/strength of the frame around the opening to be protected - in other words, can that frame support the loading to be transferred from the closure to the frame? For example, openings built into reinforced concrete floor slabs usually have extra reinforcing steel to support their perimeter. Any doubts about the adequacy of the frame to support the closure must usually be handled by some upgrading supports specifically for the frame: For example, there may need to be some timber framing around the door opening, which framing is next supported by some diagonal members blocked on the floor to other walls or to interior footings, etc., (see Figure 8 of Reference [1] for illustration of this diagonal bracing for a wall; it would be similar for a door frame or any other component requiring added support).

Frames that support the closure on two opposite sides only, call for a closure that acts as a one-way bending member that primarily experiences bending stresses along its span between the two support edges. Similarly, a closure supported on four sides is termed a two-way bending member; at the risk of being premature, passing mention is, nonetheless, made here that a two-way bending member gains little in blast resistance over a one-way member spanning the shorter span of the opening, if the longer span is more than 2 to 3 times the shorter span of the opening (more on this later). For a closure supported on three sides, the purposes of this upgrading report are best served by treating such a closure as being supported only on the two opposite sides and ignoring the support given by the third side, at least for sizing (design/analysis) purposes.

Fastening blast closures in place is a matter worth brief mention: If the closure is to resist blast (we do use closures for radiation resistance soil alone, of course), and the blast is expected to apply head-on to the plate as with a reflected blast loading, the situation is good for a closure lying on the blastward side of the frame, because there is little tendency for the closure to move sideways and it needs be held in place only to the extent required for positioning prior to blast loading. If the blast is expected to hit the closure side-on, then the fastenings would probably include cleat-type blocks on some or all sides of the closure to prevent sliding, as well as the fastenings to maintain closure position prior to blast loading. The negative blast wave that usually follows the blast wave's transit has been purposely ignored; if it's considered a problem in a specific instance, then the fastenings must be adequate to meet the negative blast phase.

Chapter 5

STRUCTURE STRENGTHENING NEEDS

This chapter deals briefly with the strengthening needs of members in the existing structure, that is the basement that has been selected for upgrading for nuclear protective shelter.

A. Wood and Steel Existing Bending Members

Wood and steel existing structure bending members have a considerable advantage for upgrading over existing reinforced concrete (R/C) members. For example, because of their material homogeneity, steel or wood beams/girders/floor stringers can have added intermediate supports at any chosen location along their length (assuming they are prismatic), whereas an existing R/C bending member can have added supports only where appropriate after considering whether the principal tensile reinforcing steel is located near the bottom or top face of the member, a serious restriction for the R/C members. One exception should be made for wood bending members, in contrast to steel members: the wood member might have been carefully placed so that a tight wood knot (falling near midspan and located near one edge of the member) is placed so that the tight wood knot is near the top (compressive) face of the member under bending stress, rather than near the bottom (tensile) face of the member under bending load; for such a case (perhaps uncommon), problems to be dealt with are similar to those in upgrading R/C bending members. Assessing the benefits of added (upgrading) interior supports under an existing wood beam, for example, does require consideration of each criterion for sizing (designing/analyzing) the member, because the mode of failure may change (from flexural to horizontal shear) in shortening the effective span of the member. For examples in wood bending member upgrading, see Appendix B2.

B. Reinforced Concrete Existing Bending Members

Reinforced concrete existing members entail much more difficulty in terms of providing added (upgrading) interior supports in beams: Perhaps the foremost problem is that of finding out how much reinforcing steel is imbedded in the concrete and where it is located. Certainly some assumptions can be made but they are an inadequate substitute for seeing a set of as-built plans, which may indeed be available from the building owner or the files of the building department of the city/county building department.

For upgrading R/C beams, assumptions can be made: The center third of a single-span, simply-supported beam will behave at least as strong as a shorter simply-supported span, if supports are added at the third points of the existing beam. This means that the blast loading resistance of the center third of the beam has been increased roughly nine times over its strength prior to upgrading. For a R/C beam continuous over several simple supports, any span might be similarly examined; if a quarter of the span length, centered on the midspan, has added (upgrading) columns, that quarter-span portion of the beam span might have its blast resistance increased roughly 16 times its strength before upgrading.

Once these easy assumptions have been made and utilized, though, things may get sticky for further upgrading, in the absence of details on the as-built reinforcing steel and its placement: However, in the beam portions between the upgraded midspan portion and the original column supports, one might put in equally spaced columns at a spacing at or less than 3 times the overall depth of the R/C beam. This approach amounts to treating each segment of the R/C beam, outside of the midspan portion just discussed, as if each new short span of such ends is a concrete pedestal (lightly reinforced). Conversely, if one has as-built details on the reinforcing steel amount and location, such things as a better approximation of lengths for the midspan portions just described, the possible membrane action of the reinforcing steel based on its imbedment lengths, etc., will allow some refinement of the location of added (upgrading) columns.

Existing structures evaluation (ESE) techniques developed by Wiehle, Bockholt, and Beck [7A,7B,7C,8] are complex and detailed, but do deal with these analysis problems peculiar to all R/C members. Similar thinking to that just described for R/C beams may be used for R/C one-way slabs, and sometimes R/C two-way slabs.

While R/C flat slabs apparently offer an upgrading potential, R/C flat plates were examined[6] for upgrading, especialy against punching shear of columns through the flat plate; the upgrading considered was to put vertical supports around each column, but even at various distances (from close to each column out to one-quarter of their span) the flat plate's predicted blast resistance was increased very little by such upgrading.

The ESE work specifically in connection with 11 selected NSS buildings scattered throughout the United States is described in Chapter 7.

Related testing and analytical work under FEMA support has been, and continues to be, done by others [9A,9B,9C], including not only the R/C member types discussed above, but also including R/C waffle slabs, flat plates, flat slabs, etc.

The following paragraphs were prepared by J. E. Beck who has just completed a report on the "as built" strength of building floor systems [8] and is currently analyzing an additional 25 buildings along with further model verification/upgrading work:

[6] For this project by C. K. Wiehle using his ESE techniques.

The best (R/C) candidate members for upgrading are those members with spans of 18 ft or greater that can be upgraded by intermediate supports at third or quarter points. Furthermore, analytical results therein indicated that if upgrading is accomplished by putting intermediate supports at about 6-ft spacing, an R/C slab about 6-in. thick can resist an overpressure of approximately 30 psi blast, and a slab about 8-in. thick can resist approximately 50 psi.

"Most R/C slabs are designed to have a span-to-thickness ratio of 24. Therefore, slabs having clear spans greater than 12 ft will normally have a potential resistance greater than 30 psi, and slabs having clear spans greater than 18 ft have a potential resistance greater than 50 psi.

"The predicted upgrading potential of R/C slabs is based on the calculated capacity of one-way simply-supported slabs with reduced spans (about 6 ft). Although it is true that the steel reinforcing in an "as-built" structure may not be at optimum points for upgrading, it is still probable that the calculated simply-supported strength of each upgraded center portion is representative of the strength of the slab. This will also be true even at ends that are designed as fixed and, therefore, may at first glance appear weaker. This statement is based on the following observations:

"(1) The detailing of a fixed joint requires twice the moment capacity of the center of the slab, and compression steel at least equal to 1/3 the central steel is present in this section.

"(2) If the joint is designed as fixed, it will act as a simply-supported beam straddling the top of the support member and will have approximately the strength of a simply-supported element having a free span equal to the clear distance between the supports."

C. Existing Columns

ESE work on R/C columns has indicated that they are adequate for much heavier blast loads than their (normal-use) design loads, and especially in the case of a multistory structure, say, 4 stories or higher, where the superstructure may be expected to allow most of the blast loading to blow through between floors rather than subject the basement columns to higher blast loadings due to such things as overturning considerations. Further, the upgrading just described for beams makes clear the plan to add a large number of columns, meaning that each original/existing column will have its contributory blast loading area on the first floor considerably reduced in size, and thus its individual column loading considerably reduced. The need, therefore, for increasing the strength of reinforced concrete columns for upgrading is small to nonexistent. Similar thinking may be applied to columns of other materials existing in the basement to be upgraded into a nuclear protective shelter, in that the remedy is simply to have existing columns share the anticipated blast loading with additional columns.

Column footings in structures to be upgraded pose no significant problems in that those under existing columns will have their load shared as just described for the columns themselves; for columns added in upgrading work, a recent test [10] at the U.S. Army Engineer Waterways Experiment Station, Vicksburg, Mississippi, involved loading a square column consisting of four 4x4s (actual 3.5x3.5 in.) strapped together and bearing on a "typical" concrete floor slab over compacted gravel over compacted soil, with the results indicating that this column could carry a static load initially peaking at 50 kips then increasing to a new peak of 84 kips and degrading somewhat, but always above 50 kips for a loading of some 10 minutes (many, many times the duration of a nuclear blast loading). This loading amounts to roughly 2,000 psi on the column and footing. At the peak loading, displacement of the footing was 1.0 in. and its displacement at the time the tests was stopped was about 1.45 in. The test will be reported in a forthcoming WES technical report, of course, but these results were gratefully received by those who have planned extensive upgrading work, where many additional columns have been planned for use supported on a "typical" basement floor slab with nothing more than a couple of wedges under the column (for erection convenience).

D. Existing Exterior Basement Walls

Exterior basement walls, whether plain CMUs, reinforced CMUs, or brick, or certainly R/C construction, have been generally accepted as having sufficient blast resistance capacity for host area shelters, and those shelters contemplated where the inherent strength of the main slab over the basement is to be exploited (say, 8-15 psi air blast peak free field overpressure). However, serious concern over the adequacy of exterior basement walls, including walls of R/C construction, has been expressed when upgrading for higher blast resistance, especially as high as 30-50 psi, is contemplated. As a result, a FEMA research technical project officer, at a recent meeting with structural research contractors at WES, included in the discussions the need for static and dynamic tests of "typical" exterior R/C walls (fully and partially buried). The purpose of the proposed tests is not to see whether design procedures apparently used were adequate (they may be conservative), but more specifically to determine the blast resistant capacity of such walls in terms of absolute ultimate behavior (through collapse), that is, to answer the question of whether or not the soil transmitting the blast loading to the exterior basement wall will indeed follow and force the wall inward beyond an acceptable displacement, closely approaching collapse. Meanwhile, planning for upgrading potential, especially for overpressure levels greater than, say, 15 or 20 psi: must be conservatively evaluated as requiring strengthening of the exterior basement walls; or can be optimistically continued on the basis that the exterior basement walls would not fail because the soil will not move fast enough to push the wall inward further than acceptable. This matter impinges seriously on the direction of Chapter 7, Shelter for Key Workers (30 to 50 psi air blast peak free field overpressure).

Chapter 6

STRUCTURE STRENGTHENING TECHNIQUES AND MATERIALS

This chapter concerns meeting the upgrading needs perceived for a particular shelter candidate basement, as described in the preceding chapter.

The chapter deals with bending members (lateral loads only) such as beams, closures, girders, etc.; columns (axial loads only); and beam-columns (both lateral and axial loads). It deals with these three kinds of structural members by sizing the members to meet the need. The sizing may be done: by determining from the need the size of member required (design), and whatever material; or by assuming a particular member's availability and finding its strength capacity (analysis), then comparing the capacity with the need. The aids available herein, as described below, may be oriented to both design and analysis depending upon which way they are entered for use. One may, for example, know the load to be met on a given floor over a basement and the contributory area to be served by a column, then look at a table of column strengths and find which column(s), if any, will meet or exceed the need, using a table/graph/chart of column strengths (thereby using a little design with a little analysis!). Above all, the reader/user should not let terminology deter him/her from using the material presented (the author hereby invites comments on any simplification needs encountered).

Appendix A has been prepared for the artisan or the professional engineer/architect as an attempt to introduce the terminology basic to the structural problems in handling impact/dynamic loads, such as air blast, meeting loads that are applied in zero to a very short time and that have a short duration (of the order of seconds).

A very useful aid in reading graphs (such as those encountered in the appendices of this report, as well as those used in such civil defense training such as that given for Fallout Shelter Analysis) may be easily prepared and will prove very useful if given a trial: obtain an ordinary drafting (plastic) 45-degree triangle, clear, of about 6 to 8 inches on the two equal (short) sides; use any straight edge, a careful eye, and any sharp pointed instrument to scribe lines parallel to each of these two sides, perhaps a quarter-inch in from the edge although this is not critical; and, use a sharp pencil to darken the bottoms of the scribed lines, then wipe off the excess graphite. The aid then can be used on a design graph where multiple curves must be either read directly or interpolated between (for a particular value described by the curves or spaces between them), in order to read values from the abscissa (bottom) and ordinate (vertical) sides of the graph.

This chapter is organized first by materials to be used (wood and steel in various forms), then by specific strengthening applications, such as the aforementioned beams (bending members), columns, and beam-columns.

Wood Availability and Use

Appendix C, Typical Stockage - Local Lumberyards, Reference [2], is short and is recommended for complete reading by the reader/user.

Specifically, the predesigns reported on below (Appendices A1 and A2) used generally available stress graded lumber in one relatively strong and one relatively weak grade and species for each of two size ranges - one pair of grades for 2x3s[7] and 2x4s (Light Framing; Construction grade for strong, and Standard grade for weak) and one pair of grades for 2x6s and 2x8s (Joists and Planks; No. 1 grade for strong, and No. 2 grade for weak). These choices are shown in Table 1B, Appendix C [2],[8] complete with the predesigns' stresses as underlined in that Table and used below (Appendices A1 and A2). Tables 1A and 2, Appendix C [2], show the complete list from which the grade choices were made. (A later complete list is provided by Table B1-2, Appendix B1 herein.)

Lumberyards have shirt-pocket size booklets, published by grading associations (list in Reference [2] of Appendix B2 herein) giving stress grading data by lumber kind, type and size; see also Reference [11].[9] In any case, try to use construction grades/stress-graded wood members; if stress data are unknown, simply use the tables/figures that follow (e.g., as in Appendices A1 and A2) and choose those calling for "lower strength stringers" or similar notation.

Although specific data have not yet been published/distributed, strong indications are appearing, among the professional engineers/architects in the research community, that lumber grading in too many areas and instances to be ignored, has not been up to standards achieved in the past; there <u>may</u> be well-founded reasons for this (the increasing proportion of second growth timber in the market), and thus not quality of work alone. At any rate, an artisan familiar with wood, even only casually, can use the shirt-pocket size grading booklets and their data, as mentioned above, plus some careful reading, to check or redo the grading work on any wood materials that become available for upgrading use. The regrading may not be as precise as that done by one regularly working in the field, but that is not the point here. Further, the artisan (e.g., one semiskilled in carpentry) need not follow upgrading guidance alone but, for an example, can see to it that, if he/she must use a wood member as a beam and must use one that has a sound knot near an edge, the sound knot is put in the top edge of the (single-span, simply-supported) beam (compressive stress) rather than in the bottom edge (tensile stress).

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[7] Nominal (actual) dimensions are 2x3 (1.5x2.5), 2x4 (1.5x3.5), 2x6 (1.5x5.5), and 2x8 (1.5x7.25) inches.

[8] Delete "Select Structural" in left column of table.

[9] Copies of References [11] and [12], one each, are furnished with each copy of this report sent to Distribution List addressees shown herein.

For plywoods, the predesigns (Appendices A1, A2, and A3) used primarily plywood grade Underlayment Interior (American Plywood Association (APA)) in Species Groups (of both face plies) #1 and #3, but the results also cover plywood grades Underlayment Exterior (APA), C-D Interior (APA), and C-C Exterior (APA), all in Species Groups #1 and #3; nominal thicknesses used were 1/2", 5/8", and 3/4". Additionally, plywood grade 2.4.1 Sturd-I-Floor Interior (APA), manufactured only in Species Group #1, was used in 1-1/8" nominal thickness, both alone and in combination with some of the thinner plywood grades just mentioned (the results also cover 2.4.1 Sturd-I-Floor Exterior (APA) used in the same thickness). All plywood grades used in predesigns are the most plentiful and were assumed to be planned for use under dry conditions (equilibrium moisture content less than 16%).[10]

Each sheet of plywood should bear an APA (American Plywood Association) Grade-Trademark stamp on one of its faces. Typical markings are shown in the fourth column of the table on page 14 of Reference [12].[9] The reader/user of this report is urged to look for and use such stamped markings, because lumberyards are prone to apply a local terminology, contrary to the formal terminology and that one stamped on the sheet of plywood; for example, a usage local to the author's home area is to advertise and describe a plywood grade as "CDX" (both orally and printed), even "CD Exterior", a plywood "grade" that is not listed in those of the APA. Instead, the plywood grade of these sheets is actually C-D Interior (APA) with exterior glue, a plywood grade that can be found in the listing of APA grades, meaning that geometric dimensions, allowable stresses, etc., can be obtained (see Reference [12][9]).

It is recommended that the stress-graded lumber user consider obtaining a copy of Reference [11].[9] Similarly but for plywood, Reference [12].[9]

A. Plywood Stressed-Skin Panels (Two-Sided)[11] as Closures

Appendix A1, Plywood Stress-Skin Panels (Two-Sided Only) as Closures - Design and Fabrication, includes a detailed treatment of the subject, aimed at the designer (engineer or architect); certain sections would be, however, of interest to the artisan reader, who might choose to gain an overview of Appendix A1 through use of its lists of Contents, Tables and Figures (pages A1-iii and -iv).

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[10] If dry conditions do not apply, all plywood must have "Exterior Glue" (so stamped); blast resistance shown in tables of Appendices A1, A2, and A3 would be reduced about 50% (conservatively; full redesigns are needed for a better estimate).

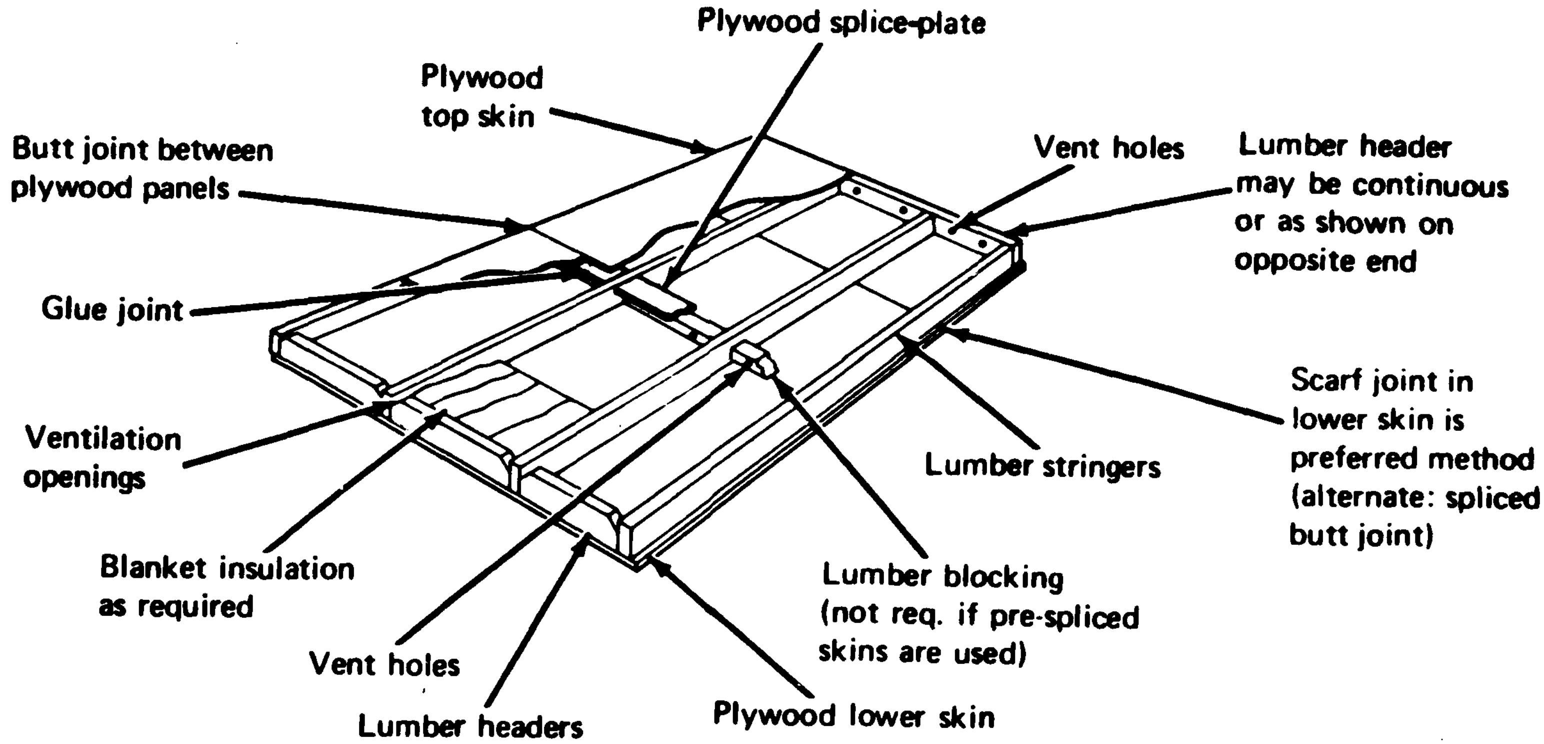
[11] Abbreviated as PSSPs herein.

Lumber ("2-by") sizes and grades considered for use in building PSSPs are discussed in the preceding section, as are plywood grades and nominal thicknesses. Figure 1 shows a cutaway perspective view of a 4-stringer PSSP, which also illustrates both continuous and non-continuous headers, as well as splice-plate installation. The latter may have little use herein, because closures are not expected to be needed in lengths longer than the 8-ft (sometimes 12-ft) plywood sheets that are available in plywood stocks of lumberyards. Another view (this one dimensioned) of a similar PSSP is shown by Figure A1-1A of Appendix A1. Both views show PSSPs with one outside stringer inset 1 in. and the other outside stringer projecting 3/4 in. so as to provide tongue-and-groove behavior among side-by-side panels; such detailing is perhaps impractical for the rapid construction of expedient option closures during a one- to three-day warning period, and such detailing is not recommended for other (mostly strength) reasons as well.

To obtain the strength benefits of stressed-skin structural behavior,[10] PSSPs are built with the plywood face plies running parallel to the stringers, which must be joined to the plywood by either nailed-glued or pressure-glued construction (the latter is better so should be used if facilities are available therefor). See the Fabrication section, Appendix A1, for details.

Table A1-1 of Appendix A1 presents 248 designs in a form for direct reading: Columns 1-4 present nominal thickness and Face Ply Species Group for the top and bottom plywood skins of the PSSP designs; columns 5-6 give the nominal size (2x4 to 2x8) and number of stringers (per 48-in. wide PSSP), with the latter showing 4, 5, 7, and 9, but 6 or 8 may be used by taking values (in columns 7 and beyond) between 5-7 and 7-9, respectively; columns 7-8 show the PSSP required bearing length on <u>each</u> end support (in addition to the clear span dimension), in terms of bearing length on the plywood bottom skin and on the stringer bottom edge, respectively (the latter happens to be controlling in all designs); the remaining columns show, for each of the designs, the PSSP's estimated peak air blast overpressure resistance (psi) for clear spans from 2 to 12 ft by half-ft increments, but omitting any value below 5 psi. In a particular application, the clear span for the blast closure will be known, as will availability (sizes, grades, etc.) of plywood and stringers; from this point one might proceed as follows:

(1) Consider that the data presented covers four plywood grades in three nominal thicknesses and two face ply species grades, plus a fifth grade in one nominal thickness and one face ply species, and the plywood combinations are each used in predesigns with two stringer strengths (termed lower and higher in Table A1-1) - all as described in the preceding section, Wood Availability and Use.



Source: Reference 1 of Appendix A1

FIGURE 1 TYPICAL TWO-SIDED PLYWOOD STRESSED-SKIN PANEL

(2) Consider the data for, say, the 6th PSSP design (with 7 stringers), Table A1-1: Top and bottom skins nominal thickness are 3/4" and 1/2", respectively, both in Face Ply Species Group #3, and the 7 (lower strength) stringers are 2x4s (nominal dimensions).

(3) Required bearing length at <u>each</u> end of the PSSP, in addition to the clear span length, would be the larger value of Columns 7-8, or 3.5 in.

(4) For a clear span of 2.5 ft, the estimated peak air blast (side-on) overpressure resistance would be 9 psi (Column 10).

(5) Equivalent free-field air blast peak overpressure to the 9 psi just found, which is for overpressure when applied side-on, would be 4 psi free-field air blast peak overpressure if applied fully reflected (i.e., head-on); the peak blast pressure felt by the PSSP would be 9 psi in either case, if taxed to its estimated design resistance. The graph on page B1-26, Appendix B1, may be used to find such "side-on" versus "head-on" equivalent free-field air blast peak overpressures, in either English or SI (metric) units (it is sufficiently accurate for purposes herein to use 1 kg/cm^2 = 100 kPa = 100 kN/m^2 in reading the graph).

(6) For PSSP widths less than 48 inches: Convert the planned PSSP width and stringer spacing to a 48-in. wide equivalent PSSP; select the equivalent 48-in. PSSP number of stringers so that its stringer spacing is equal to or wider than that in the planned PSSP. Find the applicable overpressure value for the known clear span, as above; such value may be used without reduction for PSSP widths of 24 in. or more, but it is recommended that it be reduced linearly from 0% to 50% for PSSP widths of 24" to 8", respectively, with the latter being the narrowest width recommended for use (this recommendation is adapted from Reference [2] on page A1-29). NOTE: Panel width, as used throughout this section, is always measured perpendicular to the span direction of the PSSP.

(7) Selection of a particular PSSP design for planned use would probably be by trial-and-check repeated use of Steps 2 through 6 above.

An upgrading example using the above PSSP as a horizontal closure is shown on page A1-22 and illustrates the case of apportioning the PSSP's load capacity between peak air blast loading and soil (for nuclear radiation shielding) loading.

B. <u>Plywood Stressed-Skin Panels (Two-Sided) as Columns & Beam-Columns</u>

Appendix A2 deals with this subject and includes a continuation of the design procedure of Appendix A1, thus requiring frequent reference work between the two appendices. Aside from the continuation of the design procedure, the preceding section on PSSPs as closures, and its Figure 1, should be reviewed by the reader/user, as should be certain descriptive figures and general sections of Appendix A1, perhaps especially the section on fabrication of PSSPs. Table A2-1 of Appendix A2 is a listing of the computer program developed for the design procedures in both Appendices A1 and A2, thus the program will handle design/analysis of PSSPs, whether for use as beams, columns, or beam-columns. Table A2-2 provides data on PSSP designs similar to that provided by Table A1-1 of Appendix A1, but the former has the design data for columns and beam-columns, and includes only PSSPs that have the same plywood top and bottom skins. Table A2-2 shows, for column use, the total axial dynamic/impact load (in kips, or 1000s of pounds) that each PSSP may carry, the principal (and perhaps only real) use of the data. Also shown, for beam-column use, is data on the beam-column capacity of each PSSP in terms of an axial load equal to 20%, 40%, 60%, and 80% of the pure column use load, with each percentage related to the capacity remaining for lateral loads in terms of psi. All axial loads are related to a 48 in. wide PSSP.

Appendix A2 includes a sheet, PSSP Upgrading Example - Column, showing a sample calculation for a PSSP acting as a column, axially loaded (only) with both air blast and soil for fallout radiation shielding.

C. Plywood Panels as Closures

Appendix A3, Plywood Use for Closures - Design, covers the design of plywood panels as closures over apertures such as those found in basements. The design procedure applies to panels supported on two opposite sides of an aperture, or on all four sides.

The design procedure was used to develop the predesigns shown by Tables A3-1 and -2. An example of use is as follows: Referring to Table A3-1A, assume that 3/4 in. nominal thickness plywood is available in CD-PLUGGED INTERIOR (APA) grade in Face Ply Group 3 (so stamped on each sheet of plywood), and find from the table that one should refer to Table A3-1B, Block Numbers 8 and 16, for 1/2" and 3/4" plywood, respectively, meaning that Block No. 16 applies to this example; further, one should use the second line of that Block for Face Ply Group 3, in which line one finds values of (free field, side-on, long duration) peak air blast overpressure, in psi, for seven values of clear span, in inches, such as 20 psi for an 8-inch clear span. Attention should be given to the footnote of Table A3-1B. So far span conditions dealt with have been one-way simply-supported.

Two-way (supported on all four sides) span conditions are handled by further referring to Table A3-2 where one finds that, for Block Number 16 (of Tables A3-1B), the assumed plywood would have its 20 psi/ 8-inch span strength increased by 19% if supported on all four sides of a square opening/aperture 8"x8" (1:1 ratio of longer to shorter clear spans).

Appendix A3 contains further details if desired by the user. If available plywood is not one of the 10 grades entered in Table A3-1A (including 2 multi-grade entries), recourse to the design procedure starting on page A3-3 will be necessary.

D. Wood Beams and Columns

Appendix B1, Wood Beam and Column Design - Simply Supported, covers sizing (design/analysis) of wood beams - whether used solidly (side-by-side, on edge or flatwise) or spaced as in floor joists and roof rafters, and also whether used single-span or continuous over several spans, all simply supported - as well as wood columns. Further, different treatment is shown for wood beams used solidly, when covered with a thin sheet of plywood (repetitive-member use) or without a plywood cover (single-member use), because the allowable flexural stress f_b is different for these two uses.

Appendix B1 has two major sections, Wood Beams - Simply Supported and Wood Columns - Simple Supports. A design/analysis procedure is provided in each of these two major sections.

An illustrative example is presented in paragraph (5), page B1-13, of Appendix B1, for the use of wood beams solidly side-by-side, with a thin plywood covering. The next paragraph (6) deals with the same problem, but with the thin plywood covering omitted.

Sizing of wood beams used at spacings of 12, 16, and 24 in. is covered by an illustrative example in paragraph b., page B1-14, Appendix B1. Note: The results of this sizing differs from the example shown in Appendix B2, Home Basements Upgrading in Host Areas, because the latter results are based on interior blast limited to 5 psi or less on the floor above the basement, resulting in the use of a gradual rise time; in contrast, the treatment in Appendix B1 considers the blast loading rise time as zero. The results in Appendix B2 can be converted to match those of Appendix B1 by multiplying the former by a factor of 2/3.

For correcting the blast resistance found for a simply supported beam over a single-span to a simply supported beam extending continuously over two, three, four, or five equal spans of the same individual span length, a procedure and tabulation of correction factors is provided on page B1-14, Appendix B1.

For the sizing (design/analysis) of wood columns on simple supports, a design procedure is provided beginning on page B1-15, Appendix B1. A numerical example begins on page B1-22 and uses for its solution the table and graphs of Figure B1-2. A discussion of the adequacy of end bearing in columns follows.

Review, light or heavy, of Appendix B2, Home Basements Upgrading in Host Areas, is recommended for the reader/user concerned with use of wood members in upgrading, both beams and columns; numerical examples and blast resistance values for lower and higher strength woods are included.

E. Peak Blast Resistance - Side-On versus Head-on

Finally, a conversion graph is given in Appendix B1 (page B1-26) for converting free field overpressures up to 50 psi when applied side-on, to an equal value of free field overpressure applied head-on, or fully reflected; "equal" in this sense means equal in peak blast pressure felt by the member (for example, the peak blast pressure felt by a structural member from 45 psi applied side-on is equal to 16 psi applied head-on (fully reflected)).

F. Steel Plates, Sheets, and Shapes

Appendix E1, Structural Steel Local Availability and Use for Blast Shelter Upgrading, is recommended for complete reading by the reader/user of this report.

Appendix D1, Blast-Resistant Design/Analysis of Steel Members, discusses steel design generally, structural steel material properties and strength/resistance expressions. Tables present the stresses used for sizing members (primarily plates and sheets), based on the ASTM designation for the steel, designations that are regularly used by steel suppliers. An applications section begins on page D1-9, which covers the sizing of steel plates as closures, acting either as one-way or two-way flat plates on simple supports; the section includes numerical examples carried along with the discussion. The example uses a table and graph, Figure D1-1, for solving the one-way plate sizing problem, with correction factors for all steels, and correction factors for two-way plates from the one-way plate graphical solution results.

Use of steel plates and shapes may be considerably limited by weight-handling limitations where no equipment is available, other than hand labor. (Figure D1-1 solutions stop at around 500 pounds maximum weight of the steel member.) The use of steel shapes is not covered further in this report.

Chapter 7

SHELTER FOR KEY WORKERS

There is a need for shelters upgraded to the 30 to 50 psi air blast (peak free field overpressure) range. Such shelters will be in the "risk" areas briefly mentioned on page 2 herein; this shelter is needed at locations within 15-minutes travel time from each key worker's place of work. Such shelter has also been mentioned above in paragraph (3) on page 12 where it is stated that "Work to date has indicated a low probability of getting this protection level of shelter in existing buildings, unless a truly high level of manpower and material resources is spent and, even then, serious questions need to be first answered by tests concerning the blast resistance that can be expected from existing exterior basement walls."

Also discussed earlier (paragraph D, page 18), is the very serious problem of lack of knowledge/test results on the ultimate blast resistance capacity (through collapse) of existing exterior basement walls in buildings considered for shelter use.

Table 2 presents the results of existing structures evaluation (ESE) work done by James E. Beck [8] on 11 buildings across the United States; these NSS buildings are part of the 210-building "RTI statistical sample" often used for FEMA studies, both research and operational. Table 2, presenting some results of the ESE work, also has information added under this project, as shown along the right side of each sheet of the table. Attention is particularly invited to the data on "TYPICAL BASEMENT WALL" on the right side of the first sheet of the table. the data point up the discussion of the existing exterior basement wall problem mentioned earlier and cited above. Whether one considers the mean or 1% probability value (of the incipient collapse overpressures) shown in the data, as on the right of Table 1, the overall results show very little promise for upgrading existing basements to the 30 to 50 psi blast range; whether one looks at slabs (RCS), beams (RCB), girders (RCG), or joists (RCF), of reinforced concrete, the prospects are bleak for their united floor strength exceeding a value considerably below the 30 to 50 psi range. Momentarily ignoring the basement wall problem, extensive upgrading would be required for all structural members (almost without exception) and for each closure's support frame, to reach the desired overpressure range. Cost effectiveness, in terms of expenditures of manpower and materials, could well dictate use of field type, expedient shelters, or other alternatives.

One alternative is the use of conventional large diameter pipe and culvert materials (8 ft i.d. for example), such use has been proven (by nuclear field tests) to provide high degrees of protection in a buried configuration. These include R/C pipe, corrugated steel culvert materials of round and "cattle pass" (Junior Underpass) cross sections, and arch shaped (semi circular) multi plate corrugated steel (ammunition magazine) structures [4,p 6 119 to 134]

Table 2 RESULTS FROM EXISTING STRUCTURES EVALUATION OF ELEVEN NSS BUILDINGS

11/26/79

Case	Element Type	L_e (in.)	L_t (in.)	b_b (in.)	h (in.)	f'_c (psi)	f_{dy} (psi)	Support Case	Reinforcing Ratios at Cross Sections: 1 p	1 p'	2 p	2 p'	3 p	3 p'	4 p	4 p'	Tensile Membrane Steel Ratio: Short	Long	Incipient Collapse Overpressure: Mean (psi)	Std. Dev. (psi)	10% Prob. (psi)	90% Prob. (psi)	Incipient Collapse Overpr (Tens.Membr.): Mean (psi)	Std. Dev. (psi)	1% Prob. (psi)
Building 100, U S Post Office (RCF)																									
1a	RCS-1	337	1,320		12	3,000	53,000	1	0.0172	0.0013	0.0013	0.0013	--	--	--	--	0.0013	--	5.69	0.63	4.88	6.49			
1b	RCS-1	168.5	1,320		12	3,000	53,000	5	0.0172	0.0013	--	--	--	--	--	--	0.0013	--	23.06	2.76	19.52	26.60	6.51	0.78	4.69
1c	RCS-1	112.3	1,320		12	3,000	53,000	5	0.0172	0.0013	--	--	--	--	--	--	0.0013	--	51.23	4.56	45.38	57.08	10.29	0.92	8.15
1d	RCS-1	84.25	1,320		12	3,000	53,000	5	0.0172	0.0013	--	--	--	--	--	--	0.0013	--	91.43	8.63	80.37	102.49	14.06	1.33	10.97
2a	RCS-2	223	2,320		18	3,000	53,000	5	0.0100	0.0	--	--	--	--	--	--	--	--	5.16	0.72	4.24	6.08			
2b	RCS-2	111.5	2,320		18	3,000	53,000	5	0.0100	0.0	--	--	--	--	--	--	--	--	22.72	2.83	19.09	26.35			
2c	RCS-2	74.3	2,320		18	3,000	53,000	5	0.0100	0.0	--	--	--	--	--	--	--	--	50.33	4.68	44.33	56.33			
3A	RCB-2	197		48.12	16.5	3,000	53,000	5	0.0090	0.0105	--	--	--	--	--	--	0.0150	--	28.66	3.88	23.68	33.63			
3B	RCB-2	98.5		48.12	16.5	3,000	53,000	5	0.0090	0.0105	--	--	--	--	--	--	0.0150	--	*58.71	7.05	49.67	67.76	58.71	7.05	42.30
4a	RCS-3	289	1,980		12	3,000	53,000	5	0.0139	0.0	--	--	--	--	--	--	--	--	5.83	0.62	5.03	6.62			
4b	RCS-3	144.5	1,980		12	3,000	53,000	5	0.0139	0.0	--	--	--	--	--	--	--	--	25.60	2.67	22.18	29.03			
4c	RCS-3	96.3	1,980		12	3,000	53,000	5	0.0139	0.0	--	--	--	--	--	--	--	--	59.47	6.82	50.72	68.22			
4d	RCS-3	72.25	1,980		12	3,000	53,000	5	0.0139	0.0	--	--	--	--	--	--	--	--	107.89	12.07	92.43	123.36			
5a	RCS-4	17.67	360		3	3,000	53,000	6	0.0048	0.0	--	--	0.0048	0.0	--	--	0.0048	--	*30.23	2.91	26.50	33.96			
5a/r	RCS-4	17.67	360		3	3,000	53,000	n/a	0.0048	0.0	0.0008	0.0	0.0048	0.0	0.0008	0.0	0.0048	--	143.78	12.46	127.81	159.75			
6A	RCJ-4	360		6.23	17	3,000	53,000	5T	0.0237	0.0003	--	--	--	--	--	--	--	--	3.31	0.41	2.79	3.83			
6B	RCJ-4	180		6.23	17	3,000	53,000	5T	0.0237	0.0003	--	--	--	--	--	--	--	--	14.77	1.46	12.89	16.64			
6C	RCJ-4	120		6.23	17	3,000	53,000	5T	0.0237	0.0003	--	--	--	--	--	--	--	--	35.38	4.26	29.92	40.84			
6D	RCJ-4	90		6.23	17	3,000	53,000	5T	0.0237	0.0003	--	--	--	--	--	--	--	--	62.62	7.29	53.27	71.96			
Building 110, Henry R. Landis State Hospital (RCF)																									
.1a	RCS-1	169	1,552		7	3,300	53,000	7	0.0074	0.0	--	--	0.0087	0.0	--	--	--	--	5.20	0.55	4.49	5.90			
.2A	RCB-1	264		18	20	3,300	53,000	6	0.0127	0.0	--	--	0.0182	0.0128	--	--	--	--	3.85	0.44	3.29	4.41			
.2B	RCB-1	182		18	20	3,300	53,000	7	0.0127	0.0	--	--	0.0182	0.0128	--	--	--	--	8.60	0.86	7.49	9.70			
.2C	RCB-1	88		18	20	3,300	53,000	7	0.0127	0.0	--	--	0.0182	0.0128	--	--	--	--	17.03	2.00	14.46	19.60			
.3a	RCS-2	166	254		6	3,300	53,000	3	0.0037	0.0	0.0030	0.0	--	--	0.0042	0.0027	--	--	3.31	0.49	2.69	3.94			
.4A	RCB-2	166		22	14	3,300	53,000	7	0.0093	0.0	--	--	0.0157	0.0094	--	--	--	--	6.13	0.83	5.07	7.19			
.4B	RCB-2	83		22	14	3,300	53,000	5	0.0093	0.0	--	--	--	--	--	--	--	--	14.36	1.42	12.55	16.18			
.5a	RCS-3	114	254		4	3,300	53,000	6	0.0111	0.0	--	--	0.0097	0.0056	--	--	0.0056	--	* 7.79	0.83	6.73	8.86			
.6A	RCB-3	256		20	12	3,300	53,000	6	0.0291	0.0	--	--	0.0246	0.0162	--	--	0.0126	--	* 4.33	0.63	3.52	5.14			
.6B	RCB-3	128		20	12	3,300	53,000	7	0.0291	0.0	--	--	0.0246	0.0162	--	--	0.0126	--	12.66	1.26	11.04	14.28	14.68	1.46	11.28
.6C	RCB-3	85.3		20	12	3,300	53,000	5	0.0291	0.0	--	--	--	--	--	--	0.0126	--	19.17	1.42	17.36	20.99	24.64	1.83	20.38
.6E	RCB-3	128		20	12	3,300	53,000	7	0.0291	0.0	--	--	0.0246	0.0162	--	--	0.0126	--	17.06	1.72	14.85	19.26	22.14	2.23	16.95
.7A	RCG-3	154		28	24	3,300	53,000	6	0.0245	0.0	--	--	0.0196	0.0245	--	--	--	--	5.33	0.52	4.66	5.99			
.7B	RCG-3	77		28	24	3,300	53,000	7	0.0245	0.0	--	--	0.0196	0.0245	--	--	--	--	17.94	1.83	15.60	20.28			
Building 111, Grant Building (STF)																									
.1a	RCS-1	18.33	--		2.5	2,000	44,000	6	0.0040	0.0	--	--	0.0040	0.0	--	--	0.0040	--	*14.61	0.69	13.73	15.50			
.1a/r	RCS-1	18.33	--		2.5	2,000	44,000	6	0.0040	0.0	--	--	0.0040	0.0	--	--	0.0040	--	67.59	6.87	58.79	76.40			
.2A	RCJ-1	216		5.21	12.5	2,000	44,000	7T	0.0237	0.0	--	--	0.0270	0.0205	--	--	0.0135	--	7.27	0.70	6.37	8.16			
.2B	RCJ-1	108		5.21	12.5	2,000	44,000	5T	0.0237	0.0	--	--	--	--	--	--	0.0135	--	15.13	1.44	13.28	16.98	21.40	2.04	16.66**
.2C	RCJ-1	72		5.21	12.5	2,000	44,000	5T	0.0237	0.0	--	--	--	--	--	--	0.0135	--	34.28	3.43	29.89	38.67	32.34	3.[illegible]	[illegible]4.81**
.3A	STG-1	218		12.04	16.1		41,000	6			(S=144.88 in³, 16CI83)								10.97	1.60	8.92	13.02			
.3B	STG-1	109		12.04	16.1		41,000	6			(S=144.88 in³, 16CI83)								39.25	4.37	33.65	44.85			29.08#
.3C	STG-1	72.67		12.04	16.1		41,000	6			(S=144.88 in³, 16CI83)								90.83	12.86	74.35	107.32			60.90#

TYPICAL BASEMENT WAL. Lat/vert soil press. ratio	1% Prob.
100%	6.2
50%	12
15% (unlikely value)	40

\# Steel member, thus Mean and Standard Deviation values apply unchanged from regular ESE values (i.e., no problems with beam-action stress reversals, as there are with R/C).

** Based on added girder lines at midspan (.2B) or third-points (.2C), which would increase existing girders as if upgraded (.3B and .3C, respectively). However, added girder lines would require new columns and footings/caissons carried through from five to one basements (depending on which basement(s) are to be upgraded). Even so, basement walls are non-bearing, inadequate, and infeasible to upgrade; some of first basement wall is 100% abovegrade and penetrated by garage doors.

Table 2 (continued)

Case	Element Type	L_a (in.)	L_L (in.)	b_b (in.)	h (in.)	f'_c (psi)	f_{dy} (psi)	Support Case	1 p	1 p'	2 p	2 p'	3 p	3 p'	4 p	4 p'	Tensile Membrane Steel Ratio Short	Tensile Membrane Steel Ratio Long	Incipient Collapse Overpressure Mean (psi)	Std. Dev. (psi)	10% Prob. (psi)	90% Prob. (psi)	Incipient Collapse Overpr (Tens.Membr.) Mean (psi)	Std. Dev. (psi)	1% Prob. (psi)
Building 136, First Federal Savings & Loan Assn. (RCF)																									
.1a	RCS-1	18	82		2	3,000	56,000	6	0.0036	0.0	--	--	0.0036	0.0	--	--	0.0036	--	*12.40	1.20	10.86	13.93			
.1a/r	RCS-1	18	82		2	3,000	56,000	n/a	0.0036	0.0	0.0040	0.0	0.0036	0.0	0.0040	0.0	0.0036	--	70.86	6.12	63.01	78.70			
.2A	RCJ-1	168		6.18	14	3,000	53,000	7T	0.0039	0.0039	--	--	0.0043	0.0040	--	--	--	--	3.53	0.49	2.90	4.16			
.2B	RCJ-1	84		6.18	14	3,000	53,000	5T	0.0039	0.0039	--	--	--	--	--	--	--	--	9.70	1.23	8.13	11.28			
.3A	RCG-1	228		42	14	3,000	53,000	6	0.0113	0.0113	--	--	0.0113	0.0113	--	--	--	--	4.29	0.56	3.58	5.00			
.3B	RCG-1	114		42	14	3,000	53,000	7	0.0113	0.0113	--	--	0.0113	0.0113	--	--	--	--	14.00	1.65	11.88	16.11			
.3C	RCG-1	76		42	14	3,000	53,000	5	0.0113	0.0113	--	--	--	--	--	--	--	--	20.70	2.07	18.05	23.36			
.3H	RCG-1	228		42	14	3,000	53,000	6	0.0113	0.0113	--	--	0.0113	0.0113	--	--	--	--	8.50	0.83	7.44	9.57			
Building 167, Lafayette Towers Building 2 (RCF)																									
.1a	RCFP-1	240	240		10	3,000	53,000	11	0.0030	0.0	0.0030	0.0015	0.0069	0.0	0.0108	0.0070	--	--	6.31	0.76	5.33	7.29			
.1a/s	RCFP-1	240	240		10	3,000	53,000	11	0.0030	0.0	0.0030	0.0015	0.0069	0.0	0.0108	0.0070	--	--	1.63	0.01	1.62	1.64			
.1h	RCFP-1	120	120		10	3,000	53,000	11	0.0030	0.0	0.0030	0.0015	0.0069	0.0	0.0108	0.0070	--	--	30.83	3.96	25.75	35.91			
.1h/s	RCFP-1	120	120		10	3,000	53,000	11	0.0030	0.0	0.0030	0.0015	0.0069	0.0	0.0108	0.0070	--	--	8.77	0.02	8.75	8.79			
.1i	RCS-1	240	240		10	3,000	53,000	2	0.0030	0.0	0.0030	0.0	0.0030	0.0	0.0030	0.0015	--	--	9.13	0.98	7.87	10.39			
.1j	RCS-1	120	120		10	3,000	53,000	1	0.0030	0.0	0.0030	0.0	--	--	--	--	--	--	19.90	2.59	16.57	23.22			
Building 188, State Wildlife Conservation Building (RCF)																									
.1a	RCS-1	192	--		6	3,000	44,000	6	0.0094	0.0	--	--	0.0140	0.0025	--	--	0.0094	--	* 9.95	1.07	8.58	11.32			
.1b	RCS-1	96	--		6	3,000	44,000	7	0.0094	0.0	--	--	0.0140	0.0025	--	--	0.0094	--	*20.81	2.34	17.81	23.81	20.81	2.34	15.36
.2A	RCB-1	157.5		48	15.5	3,000	44,000	7	0.0086	0.0	--	--	0.0126	0.0034	--	--	0.0051	--	7.15	0.96	5.92	8.39			
.2B	RCB-1	78.75		48	15.5	3,000	44,000	5	0.0086	0.0	--	--	0.0126	0.0034	--	--	0.0051	--	13.59	1.71	11.40	15.79	11.47	1.44	8.11
.3A	RCB-1	233.5		48	15.5	3,000	44,000	7	0.0131	0.0	--	--	0.0126	0.0049	--	--	0.0080	--	* 3.36	0.58	2.61	4.10			
.3B	RCB-1	116.75		48	15.5	3,000	44,000	5	0.0131	0.0	--	--	--	--	--	--	0.0080	--	8.85	0.95	7.64	10.86	12.97	1.39	9.73
.3C	RCB-1	77.83		48	15.5	3,000	44,000	5	0.0131	0.0	--	--	--	--	--	--	0.0080	--	21.49	2.53	18.24	24.73	19.84	2.34	14.40
Building 200, Fitzsimmons General Hospital (RCF)																									
.1a	RCS-1	78	266		4	3,000	44,000	6	0.0073	0.0	--	--	0.0073	0.0073	--	--	--	--	6.54	0.54	5.85	7.22			
.1e	RCS-1	78	133		4	3,000	44,000	4	0.0073	0.0	0.0040	0.0	0.0073	0.0073	--	--	--	--	8.77	0.83	7.71	9.84			
.2A	RCB-1	266		12	18	3,000	44,000	6	0.0138	0.0	--	--	0.0160	0.0084	--	--	0.0138	--	* 4.79	0.56	4.07	5.52			
.2B	RCB-1	133		12	18	3,000	44,000	7	0.0138	0.0	--	--	0.0160	0.0084	--	--	0.0138	--	13.86	1.67	11.72	15.99	18.96	2.28	13.64
.3A	RCG-1	255		16	28	3,000	44,000	6	0.0101	0.0	--	--	0.0074	0.0076	--	--	0.0025	--	4.92	0.52	4.25	5.60			
.3B	RCG-1	127.5		16	28	3,000	44,000	7	0.0101	0.0	--	--	0.0074	0.0076	--	--	0.0025	--	17.09	2.15	14.32	19.85	3.25	0.41	2.29
.3C	RCG-1	85		16	28	3,000	44,000	5	0.0101	0.0	--	--	--	--	--	--	0.0025	--	28.50	3.46	24.07	32.93	5.16	0.63	3.70
.3H	RCG-1	255		16	28	3,000	44,000	6	0.0101	0.0	--	--	0.0074	0.0076	--	--	0.0025	--	8.55	0.96	7.31	9.78	2.45	0.28	1.81
.4a	RCS-2	108	180		4	3,000	44,000	6	0.0073	0.0	--	--	0.0073	0.0073	--	--	--	--	3.28	0.30	2.90	3.66			
.4e	RCS-2	90	108		4	3,000	44,000	3	0.0040	0.0	0.0073	0.0	--	--	0.0073	0.0073	--	--	6.06	0.73	5.13	6.99			
.5A	RCB-2	177		12	16	3,000	44,000	6	0.0086	0.0	--	--	0.0053	0.0085	--	--	0.0086	--	* 3.94	0.62	3.15	4.73			
.5B	RCB-2	88.5		12	16	3,000	44,000	7	0.0086	0.0	--	--	0.0053	0.0085	--	--	0.0086	--	11.99	1.16	10.50	13.47	4.54	1.41	11.27
.5E	RCB-2	88.5		12	16	3,000	44,000	7	0.0086	0.0	--	--	0.0053	0.0085	--	--	0.0086	--	18.65	2.23	15.80	21.51	21.77	2.60	15.71
.6A	RCB-2	105		12	20	3,000	44,000	6	0.0041	0.0029	--	--	0.0029	0.0041	--	--	0.0070	--	12.17	0.74	11.22	13.12			

Table 2 (continued)

Case	Element Type	L_s (in.)	L_L (in.)	b_b (in.)	h (in.)	f'_c (psi)	f_{dy} (psi)	Support Case	Reinforcing Ratios at Cross Sections: 1 p	1 p'	2 p	2 p'	3 p	3 p'	4 p	4 p'	Tensile Membrane Steel Ratio: Short	Long	Incipient Collapse Overpressure: Mean (psi)	Std. Dev. (psi)	10% Prob. (psi)	90% Prob. (psi)	Incipient Collapse Overpr (Tens.Membr.): Mean (psi)	Std. Dev. (psi)	1% Prob. (psi)
Building 220, Fidelity Federal Plaza Building (RCF)																									
.1a	RCS-1	95	306		4.5	3,000	53,000	6	0.0043	0.0	--	--	0.0043	0.0021	--	--	0.0043	--	* 8.30	1.08	6.92	9.68			
.1e	RCS-1	95	153		4.5	3,000	53,000	4	0.0043	0.0	0.0037	0.0	0.0043	0.0021	--	--	0.0043	--	* 8.99	0.99	7.72	10.26	8.99	0.99	6.69
.1f	RCS-1	95	102		4.5	3,000	53,000	4	0.0043	0.0	0.0037	0.0	0.0043	0.0021	--	--	0.0043	--	* 9.80	0.98	8.55	11.06	9.80	0.98	7.52
.2A	RCB-1	306		13	20	3,000	53,000	6	0.0137	0.0006	--	--	0.0172	0.0103	--	--	0.0137	--	* 5.50	0.84	4.42	6.58			
.2B	RCB-1	153		13	20	3,000	53,000	7	0.0137	0.0006	--	--	0.0172	0.0103	--	--	0.0137	--	15.17	1.82	12.84	17.50	20.36	2.44	14.68
.2C	RCB-1	102		13	20	3,000	53,000	5	0.0137	0.0006	--	--	--	--	--	--	0.0137	--	20.21	1.81	17.89	22.52	30.85	2.76	24.42
.2D	RCB-1	76.5		13	20	3,000	53,000	5	0.0137	0.0006	--	--	--	--	--	--	0.0137	--	36.44	3.25	32.28	40.60	41.35	3.69	32.77
.3A	RCG-1	312		18	36	3,000	53,000	6	0.0063	0.0063	--	--	0.0063	0.0063	--	--	0.0078	--	3.72	0.36	3.26	4.18			
.3B	RCG-1	156		18	36	3,000	53,000	7	0.0063	0.0063	--	--	0.0063	0.0063	--	--	0.0078	--	12.21	1.25	10.60	13.81	11.19	1.15	8.51
.3C	RCG-1	104		18	36	3,000	53,000	5	0.0063	0.0063	--	--	--	--	--	--	0.0078	--	20.82	2.12	18.11	23.54	19.10	1.94	14.57
.3D	RCG-1	78		18	36	3,000	53,000	5	0.0063	0.0063	--	--	--	--	--	--	0.0078	--	34.84	4.42	29.17	40.51	23.01	2.92	16.21
.3H	RCG-1	312		18	36	3,000	53,000	6	0.0063	0.0063	--	--	0.0063	0.0063	--	--	0.0078	--	7.90	0.94	6.69	9.11	10.96	1.30	7.93
Building 225, Broadway Crenshaw Building (RCF)																									
.1a	RCFP-1	288	288		9	2,500	53,000	11	0.0044	0.0	0.0054	0.0	0.0060	0.0	0.0094	0.0	0.0027	0.0027	* 7.17	0.99	5.90	8.44			
.1a/s	RCFP-1	288	88		9	2,500	53,000	11	0.0044	0.0	0.0054	0.0	0.0060	0.0	0.0094	0.0	0.0027	0.0027	* 6.88	0.75	5.92	7.85			
.1i	RCS-1	288	288		9	2,500	53,000	2	0.0044	0.0	0.0044	0.0	0.0044	0.0	0.0054	0.0	0.0027	0.0027	* 8.79	0.99	7.70	10.23	8.79	0.99	6.49
.1j	RCS-1	144	144		9	2,500	53,000	1	0.0044	0.0	0.0044	0.0	--	--	--	--	0.0027	0.0027	*18.22	2.56	14.94	21.49	18.22	2.56	12.26
.2a	RCS-2	76	286		4	2,500	53,000	5	0.0111	0.0	--	--	--	--	--	--	0.0056	--	*11.49	1.45	9.62	13.35			
.3A	RCB-2	286		16	25	2,500	53,000	7	0.0123	0.0	--	--	0.0126	0.0062	--	--	0.0062	--	7.46	0.94	6.25	8.66			
.3B	RCB-2	143		16	25	2,500	53,000	5	0.0123	0.0	--	--	--	--	--	--	0.0062	--	19.88	1.96	17.38	22.39	16.16	1.59	12.45
.3C	RCB-2	95.3		16	25	2,500	53,000	5	0.0123	0.0	--	--	--	--	--	--	0.0062	--	46.94	5.27	40.19	53.70	24.63	2.76	18.19
.4A	RCG-2	253		26	30	2,500	53,000	7	0.0156	0.0	--	--	0.0233	0.0081	--	--	0.0078	--	12.46	1.43	10.62	14.29			
.4B	RCG-2	126.5		26	30	2,500	53,000	5	0.0156	0.0	--	--	--	--	--	--	0.0078	--	30.09	3.18	26.02	34.17	17.97	1.90	13.55
.4C	RCG-2	84.3		26	30	2,500	53,000	5	0.0156	0.0	--	--	--	--	--	--	0.0078	--	68.62	7.05	59.59	77.66	27.29	2.80	20.77
.4H	RCG-2	253		26	30	2,500	53,000	7	0.0156	0.0	--	--	0.0233	0.0081	--	--	0.0078	--	25.26	2.90	21.54	28.98	17.68	2.03	12.96
Building 227, May Company Eastland Shopping Center (RCF)																									
.1a	RCS-1	29			3	3,000	56,000	6	0.0032	0.0	--	--	0.0032	0.0	--	--	0.0032	--	*10.19	0.66	9.35	11.03			
.1a/r	RCS-1	29			3	3,000	56,000	n/a	0.0032	0.0	0.0032	0.0	0.0032	0.0	0.0032	0.0	0.0032	--	51.50	5.25	44.77	58.24			
.2A	RCJ-1	270		7.5	17	3,000	53,000	7T	0.0107	0.0	--	--	0.0107	0.0053	--	--	0.0053	--	3.49	0.40	2.98	4.00			
.2B	RCJ-1	135		7.5	17	3,000	53,000	5T	0.0107	0.0	--	--	--	--	--	--	0.0053	--	10.20	1.37	8.44	11.96	10.60	1.43	7.30
.2C	RCJ-1	90		7.5	17	3,000	53,000	5T	0.0107	0.0	--	--	--	--	--	--	0.0053	--	21.91	2.57	18.62	25.21	16.19	1.90	11.77
.3A	RCG-1	282		30	24	3,000	53,000	6	0.0157	0.0	--	--	0.0189	0.0079	--	--	0.0137	--	7.27	0.87	6.15	8.39			
.3B	RCG-1	141		30	24	3,000	53,000	7	0.0157	0.0	--	--	0.0189	0.0079	--	--	0.0137	--	21.30	2.06	18.66	23.94	20.37	1.97	15.79
.3C	RCG-1	94		30	24	3,000	53,000	5	0.0157	0.0	--	--	--	--	--	--	0.0137	--	31.80	3.82	26.90	36.69	30.79	3.70	22.19
.3H	RCG-1	282		30	24	3,000	53,000	6	0.0157	0.0	--	--	0.0189	0.0079	--	--	0.0137	--	14.31	1.35	12.59	16.04	20.15	1.90	15.73
.4A	RCG-1	280		30	26	3,000	53,000	6	0.0118	0.0	--	--	0.0156	0.0	--	--	--	--	6.73	0.65	5.90	7.56			
.4B	RCG-1	140		30	26	3,000	53,000	7	0.0118	0.0	--	--	0.0156	0.0	--	--	--	--	20.87	2.35	17.86	23.88			
.4C	RCG-1	93.3		30	26	3,000	53,000	5	0.0118	0.0	--	--	--	--	--	--	--	--	28.90	3.28	24.69	33.10			
.4H	RCG-1	280		30	26	3,000	53,000	6	0.0118	0.0	--	--	0.0156	0.0	--	--	--	--	14.23	1.63	12.13	16.32			

Table 2 (concluded)

Case	Element Type	L_s (in.)	L_L (in.)	b_b (in.)	h (in.)	f'_c (psi)	f_{dy} (psi)	Support Case	Reinforcing Ratios at Cross Sections: 1 p	1 p'	2 p	2 p'	3 p	3 p'	4 p	4 p'	Tensile Membrane Steel Ratio: Short	Long	Incipient Collapse Overpressure: Mean (psi)	Std. Dev. (psi)	10% Prob. (psi)	90% Prob. (psi)	Incipient Collapse Overpr (Tens.Membr.): Mean (psi)	Std. Dev. (psi)	1% Prob. (psi)
Building 245, Portland Hilton Hotel																									
.1a	RCS-1	24	112		3	3,000	56,000	6	0.0029	0.0	--	--	0.0029	0.0	--	--	0.0029	--	14.45	0.08	14.34	14.56			
.1a/r	RCS-1	24	112		3	3,000	56,000	n/a	0.0029	0.0	0.0005	0.0	0.0029	0.0	0.0005	0.0	0.0029	--	97.30	9.77	84.78	109.83			
.2A	RCJ-1	270		5.5	15	3,000	53,000	6T	0.0181	0.0103	--	--	0.0275	0.0106	--	--	--	--	8.49	0.86	7.38	9.59			
.2B	RCJ-1	135		5.5	15	3,000	53,000	7T	0.0181	0.0103	--	--	0.0275	0.0106	--	--	--	--	25.79	3.18	21.71	29.86			
.2C	RCJ-1	90		5.5	15	3,000	53,000	5T	0.0181	0.0103	--	--	--	--	--	--	--	--	32.78	4.12	27.50	38.05			
.3A	RCG-1	936		30	76.8	3,000	53,000	6	0.0221	0.0110	--	--	0.0221	0.0184	--	--	0.0147	--	11.26	1.56	9.26	13.27			
.3B	RCG-1	468		30	76.8	3,000	53,000	7	0.0221	0.0110	--	--	0.0221	0.0184	--	--	0.0147	--	35.04	3.75	30.23	39.85	7.59	0.81	5.70
.3C	RCG-1	312		30	76.8	3,000	53,000	5	0.0221	0.0110	--	--	--	--	--	--	0.0147	--	53.65	6.50	45.31	61.98	11.90	1.44	8.55
.3D	RCG-1	234		30	76.8	3,000	53,000	5	0.0221	0.0110	--	--	--	--	--	--	0.0147	--	92.98	8.69	81.84	104.12	16.22	1.52	12.69

The question arises, however, as to how much of these materials could be mobilized and installed in a crisis build-up period of, say, one year.

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Appendix A

BLAST-RESISTANT DESIGN/ANALYSIS GENERAL APPROACH

CONTENTS

FIGURES

Appendix A

BLAST-RESISTANT DESIGN/ANALYSIS GENERAL APPROACH

Introduction

The purpose of this appendix is to discuss several matters that are common to all blast-resistant design/analysis approaches, whatever th. construction material. This and the following engineering appendices are aimed at architects and engineers, but generally those weak in, say, indeterminate structures and dynamic structural analysis.

For collateral reading, there are many references: the overall source-book on nuclear weapons effects [1, using its Contents and Index to locate items of interest],[1] an early paper, tightly written, that is now a classic [2] (an errata was later published: delete the denominator "6" in Eq. 2, p. 49; delete the fraction bar in text line below Eq. 7a, p. 56; and, change the "q" subscript to "e" in line 7, p. 58); the first professional society manual on the subject, still current [3]; two bulky manuals [4,5]; a textbook [6]; and, two publications used in the preparation of this and other appendices herein [7,8].

In this and following appendices, design and analysis may be used with little difference in meaning. In most cases dealt with in upgrading, design is accomplished by seeing what materials one has to work with, then determining each potential member's blast resistance (analysis) when used under a particular set of conditions. Most often a design/analysis procedure will be presented followed by a simplifying graph or table, which is used by entering either with the member parameters to find its blast resistance, or with its desired blast resistance to find the needed member parameters (size, useful stresses, span, etc.).

Newmark has clearly stated a simplified approach to blast-resistant design/analysis [8(Ch.7)] from which the following material is quoted (parenthetic connective words or insertions are by this writer):

"The various factors governing structural design for blast resistance . . . include loading, structural resistance, design considerations as affected by the material used, and the structural properties of these materials. . . . Resistance expressions for different materials and support conditions (are presented in following appendices).

---

[1] Brackets are used herein to indicate sources in the References list at the end of this appendix.

Loading

The simplified loading (Figure A-1) of the various structural elements . . . are . . . assumed (to be) . . . long duration step pulses[2] with pressures p_m, which are a function of the location of the elements and their orientation with respect to the blast wave. (In Figure A-1:

t = time, measured from arrival of the blast front (sec).)

Structural Yield Resistance

"For the long duration step pulse loading shown in Figure A-1 the relationship (among) the design parameters is:

$$p_m/q_y = 1 - 1/(2\mu) \tag{1}$$

where:

p_m = peak pressure (psi)

p_y = the (idealized) yield resistance of the structural element (psi)

μ = the ductility factor defining the maximum acceptable response of the structure, i.e., the ratio of the maximum deflection to the yield deflection

"Two important design parameters do not appear in the above equation. These parameters are:

t_d = the duration of the (positive phase of the blast) loading (sec)

T = the (effective natural) period (of vibration) of the structural element (sec)

These parameters do not appear (in Eq. 1) because the ratio t_d/T is considered to be infinite, or effectively greater than about 3. Reference [2 or 3] may be consulted to obtain the relationships replacing the above equation when the ratio t_d/T is small.

"The structural yield resistance q_y and the limit of acceptable structural response μ are determined by considering the resistance-deflection curve for the structural element. The resistance q (is) considered as a static loading distributed spatially in the same manner as the air blast loading;

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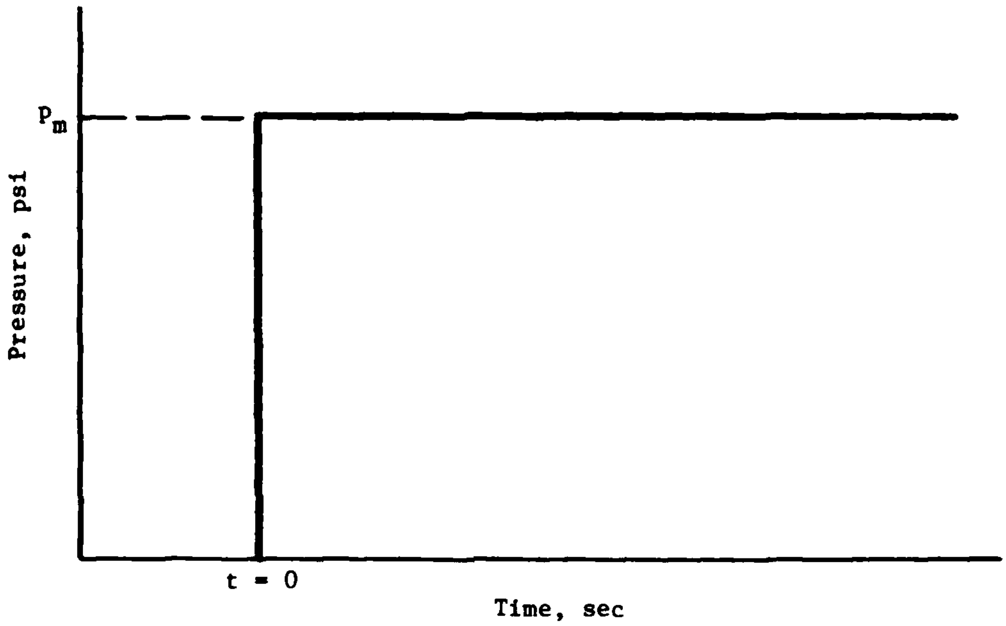
[2] A loading whose application is instantaneous (i.e., rise time equals zero) and duration is long.



Source: Reference 8, page 169

Figure A-1 BLAST LOADING

q = structural resistance (psi)

is plotted in Figure A-2 , where

q_y = the structural yield resistance (psi)

The deflection is also plotted in Figure A-2, where

x_e = the yield deflection.

The ductility factor

μ = (x_m/x_e)

where

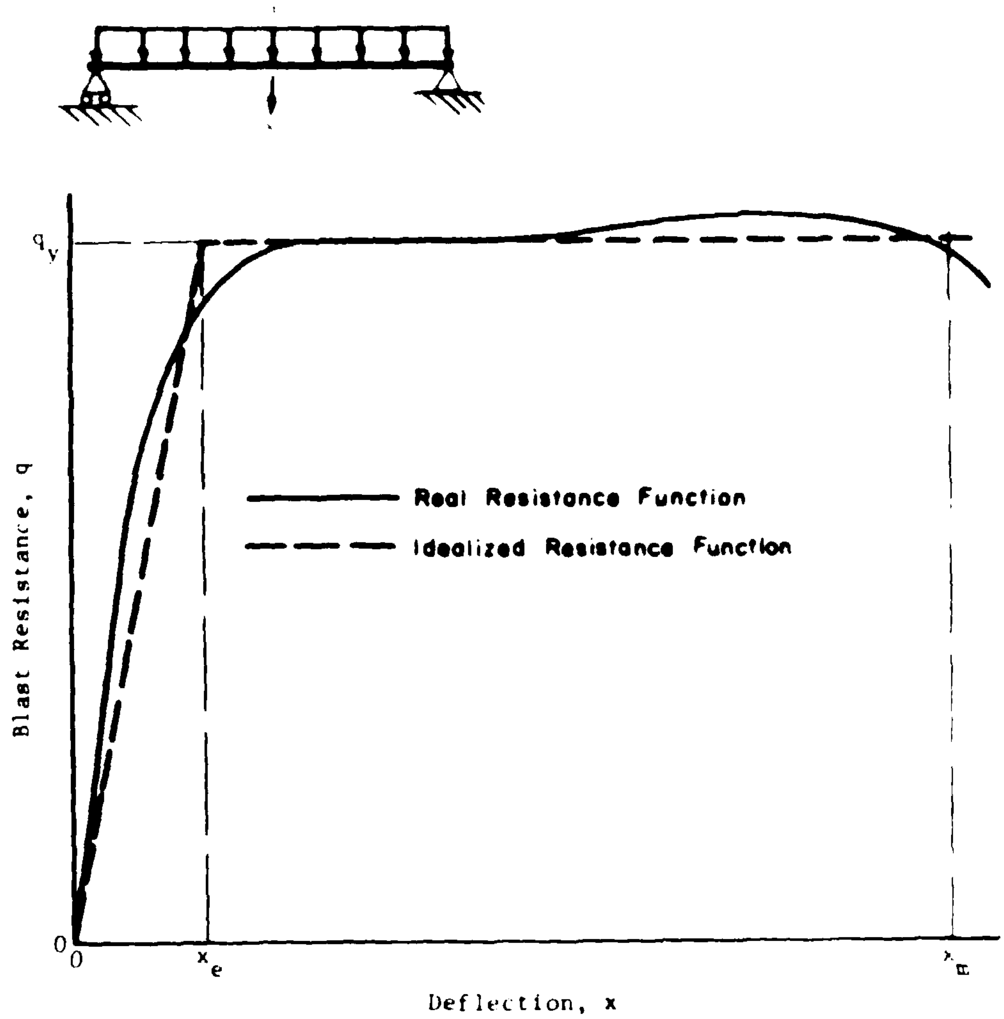
x_m = the maximum acceptable deflection.

In the selection of x_m, both structural integrity and structural function should be considered. The value of x_m should not be greater than the deflection at which the resistance of the structure begins to drop off or fracture occurs. Operational requirements, e.g., avoidance of jamming of a door or its operating mechanism, may set a lower limit on x_m.

"Figure A-2 is a typical resistance-deflection curve with its idealized bi-linear representation as used in design. The idealized resistance function is constructed so that the area under both the real and ideal curves are equal (from zero to) yield (x_e) and (from yield to) maximum response (x_m).

"The design involves establishing the required yield resistance for the structural element and then providing this resistance in the structural element (or analysis involves knowing the latter and finding q_y). The peak pressure p_m is evaluated (by considering expected peak overpressure, peak expected pressure and reflection factor, peak room filling pressure [7(p.8-112)], etc., and) the orientation of the structural element with respect to the blast wave. If allowable maximum deflection is set by structural considerations, the ductility factor μ is a function of the type of structural element and materials used. If operational requirements govern, the ductility factor μ will be selected to limit the maximum deflection of the structure to the permissible magnitude.

"In the (following appendices), expressions are presented for yield resistance q_y, yield deflection x_e, and the period T for (structural) elements of various materials and structural types. The maximum recommended value of the ductility factor based on consideration of structural integrity is presented for each material and each structural type considered. These μ values must then be checked to insure that the resulting deformations are operationally acceptable.



$\mu = x_m / x_e$

Source: Adapted from Reference 8, page 170

Figure A-2 BLAST RESISTANCE

(Typical Real and Idealized Load-Deflection Curves)

Structural Properties of Materials

"In following appendices are presented the strength properties of materials considered suitable for (basement protective) structures. In many instances the variety of commercially available materials is too great for complete description of pertinent properties herein. Therefore expressions for design stresses for protective design (may be) given in lieu of extensive strength tabulations.

"Protective structures are designed on the basis of a predicted failure load, failure being defined either by the limit of acceptable deformation or by the (incipient) collapse of the structural element. Normally a ductile (large deformation) failure mode is desired in design since large amounts of energy are absorbed in inelastic deformation.

"The design stresses given herein correspond generally to the probable yield stress of the material under the blast loading conditions. These design stresses represent probable yield stresses for the material, not guaranteed minimum values, since it is desired to estimate the actual yield load for the structural element rather than a lower limit to the yield load."

Design versus Weapons Analyst Overpressure

One must emphasize that the peak blast <u>design overpressure</u> p_m is not a weapons analyst overpressure. The former is a value that should give, say, a 99% probability of not reaching incipient collapse or some other failure definition, whereas the latter should be a median probability value [illegible] p. 6-2[illegible]]

Factor of Safety

Blast resistant design/analysis (for, say, 99% probability of not reaching incipient collapse or some other failure definition) is based on a peak blast pressure value that has all factors of safety wrung out of it. In structural design to resist dynamic loads, various structural elements are likely to have different dynamic response characteristics (e.g., different natural period of elastic vibration T). Thus, if all structural elements are to be designed to the same dynamic strength level, the loading must be in terms of a dynamic load(s). Therefore, if a factor of safety, common to all structural elements is to be set, it must be in terms of the dynamic loading, that is, in this case air blast peak pressure. Because protective structures against nuclear air blast are very expensive, the dynamic factor of safety is usually taken as unity, i.e., the peak air blast pressure (1) considered most likely to occur or (2) the one assumed for design use as the pressure that one can afford to protect against. If the latter is to be subject to a further factor of safety, for example two, then this factor is simply used as a multiplier on the peak pressure arrived at by either of the two methods just described.

NOTATION

p_m = peak pressure (psi)

p_y = the (idealized) yield resistance of the structural element (psi)

q = structural resistance (psi)

q_y = the structural yield resistance (psi)

T = the (effective natural) period (of vibration) of the structural element (sec)

t = time, measured from arrival of the blast front (sec)

t_d = the duration of the (positive phase of the blast) loading (sec)

x_e = the yield deflection

x_m = the maximum acceptable deflection

μ = (x_m/x_e)

μ = the ductility factor defining the maximum acceptable response of the structure, i.e., the ratio of the maximum deflection to the yield deflection

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[3] Those references for which "AD-" numbers are shown are understood to be available for purchase from NTIS, Springfield, Virginia, 22151.

[4] Reprinted, without change, and redesignated; e.g., EM 1110-345-413 through EM 1110-345-421 became Headquarters, Department of the Army, TM 5-856-1 through TM 5-856-9.

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[6] Now Federal Emergency Management Agency

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Appendix A1

PLYWOOD STRESSED-SKIN PANELS (TWO-SIDED ONLY) AS CLOSURES -

DESIGN AND FABRICATION

Extracts (with minor revisions) from the main text and Appendix A1 of

Murphy, H. L., Upgrading Basements for Combined Nuclear Weapons Effects: Predesigned Expedient Options, Stanford Research Institute* Technical Report, for U.S. Defense Civil Preparedness Agency,# October 1977. (AD-A054 409)

* Now SRI International
\# Now Federal Emergency Management Agency

CONTENTS

TABLE

FIGURES

Design

A design procedure for plywood stressed-skin panels was developed because plywood and suitable wood members for the necessary stringers are in abundant supply in local lumberyards, and because efficient use of such materials can assist greatly in meeting the existing basement upgrading need for many closures against air blast entry into the basement.

Existing design procedures were studied, used as a basis for developing the procedure that follows, but had to be carefully reviewed/modified/rederived to make them both dimensionally consistent (and thus more readily convertible to metric units, a contract requirement) and usable for panel widths other than 48 in. (a limitation built into the present procedure).[1,2*]

The developed design procedure is limited to plywood stressed-skin panels with both top and bottom skins, both of which are used with the grain of the outer plies parallel to the stringers. Adequate shear transfer between plywood (flanges) and stringers (webs) is assumed, based on using pressure-glued or nail-glued joining techniques. The normal-use allowable stresses in the procedure are intended for application to panels at least 2 ft wide (measured perpendicular to stringers); narrower panels are subject to reductions in allowable stresses.[2(p.19)]

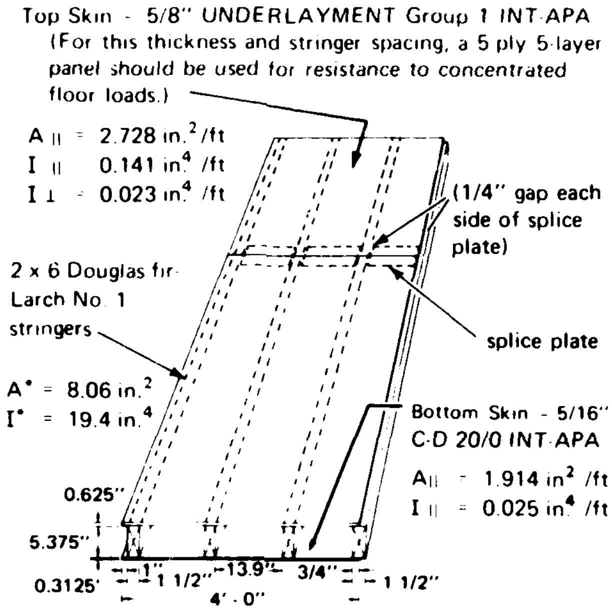
Design Procedure. The design procedure (steps) follows:

1. Assume a trial section and clear span (in direction of stringers), and that panel is fully and uniformly loaded. See Figure 1A.

2. Get values for b ("b distance"), both for top b_t and bottom b_b skins (Figure 1B). If clear distance between stringers, Figure 1A, exceeds 2b for both skins, this design procedure is inapplicable.[1(p.5)]

3. Calculate N.A. (neutral axis location) for deflection. Use bottom of panel as reference line for moment arms y applied to areas $A_{//}E$,

* Superscript numbers are related to the references list at the end of this appendix. Reference 2 must be held by the user, particularly for its Tables, pp. 9, 14-17 and 26; holding Reference 1 is unnecessary but may be desirable. (Reference of this report's main text is later reference and not significantly changed from Reference 2 herein.)



$$\text{Clear distance between stringers} = \frac{48 - 3 \times 1.5 - 1 - 0.75}{3} = 13.9''$$

$$\text{Total splice plate width} = 3(13.9 - 0.5) = 40.2''$$

*Includes a 1/8" reduction in depth to allow for resurfacing.

A.

Basic Spacing, *b*, For Various Plywood Thicknesses
(Face grain parallel to stringers*)

Plywood	Basic Spacing, *b* (inches) Number of plies				
	3	4 (3 layer)	5 (5 layer)	6 (5 layer)	7 (7 layer)
1/4" Sanded	10				
5/16" Unsanded	12				
3/8" Unsanded	16				
3/8" Sanded	19				
1/2" Unsanded, sanded, touch sanded	22	22	23		
5/8" Unsanded, sanded	27	35	33		
5/8" 19/32" Touch sanded		27	32		
3/4" Unsanded, sanded, touch sanded		36	38	38	
23/32" Touch sanded		35	34	37	
7/8" Unsanded			48		39
7/8" Sanded					51
1" Unsanded, sanded					53
2·4·1					56

*Where plywood face grain is across stringers, write APA for appropriate "*b*" distances.

B.

FIGURE A1-1 PLYWOOD STRESSED-SKIN PANEL (Example Trial Section)[1(p.4)] AND TABLE ON STRINGER SPACING[1(p.5)] counting only plies parallel to stringers (for $A_{//}$) and increasing E values (to correct from effective E to true E in bending), by 10% for skins[2(p.17),1(p.7)] and 3% for stringers.[3,1(p.7)] $A_{//}$ values are available from tables[2(p.16: note units, col. 4: in.²/ft)]. A calculation example is shown in Figure 2A.

4. Calculate panel (EI_g) using N.A. of Step 3. This stiffness factor is for moment deflection only (i.e., excludes shear deflection). Obtain I_o values for skins.[2(p.16,col.5)] Calculate I_o values for (combined) stringers ($bd^3/12$), including a portion of any stringer that is partially outside the plywood skins, as one stringer is in the calculation example shown in Figure 2B. Same E values and percentage increases are used as in Step 3.

5. Calculate allowable load (TL) - deflection:*†

$$p_d = 1 \,/\, [C\ell\ell' \left(\frac{5}{384} \frac{\ell^2}{(EI_g)} + \frac{0.15}{AG} \right)] \quad + \text{DL}$$

where: p_d = allowable TL - panel deflection (psi)

C = factor for max. allowable deflection*
(often 360 floors, 240 roofs, LL only)

(EI_g) from Step 4 (lb-in.^{2})

A = (actual) total X-sec. area of all stringers (in.^{2})

G = modulus of rigidity of stringers (psi)
(taken as 0.06 E plus 3%)

ℓ = clear span of panel (in direction of stringers)(in.)

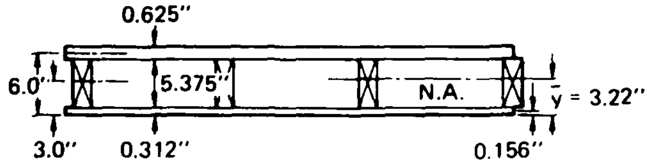
ℓ' = width of panel (skins only)(perpendicular to ℓ)(in.)

6. Calculate allowable load (TL) - top skin deflection (cross-panel). Usually only the top skin deflection need be checked, but unusual assumed sections may require top skin moment and shear investigations.[1(p.9)] Check strip 1 in. wide for allowable total load (TL) and deflection

---

* If C is based on TL, then p_d will be directly in TL units (psi), without adding the DL term in the equation.[1(p.9)]

† While (EI_g) excludes shear deflection, the formula for p_d includes it.

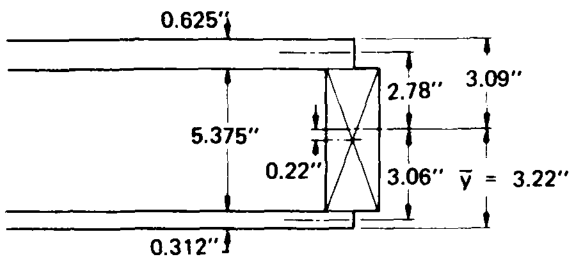


Values of $A_{\parallel}$ of plywood from PDS Table 1.

Item	E	$A_{\parallel}$	$A_{\parallel}E$	y	$A_{\parallel}Ey$
Top Skin	1,800,000 x 1.1 = 1,980,000	4 x 2.728 = 10.9	21,600,000	6.000	129,600,000
Stringers	1,800,000 x 1.03 = 1,850,000	4 x 8.06 = 32.2	59,600,000	3.000	178,800,000
Bottom Skin	1,800,000 x 1.1 = 1,980,000	4 x 1.914 = 7.66	15,200,000	0.156	2,370,000
Total		50.8	96,400,000		310,770,000

$$\bar{y} = \frac{\Sigma A_{\parallel}Ey}{\Sigma A_{\parallel}E} = \frac{310,770,000}{96,400,000} = 3.22''$$

A.



Values of I_o of plywood from PDS Table 1.

Item	E	I_o	$A_{\parallel}$	d	d^2	$A_{\parallel}d^2$	$I_o + A_{\parallel}d^2$	$E(I_o + A_{\parallel}d^2)$
Top Skin	1,980,000	0.564	10.9	2.78	7.73	84.3	84.9	168,000,000
Stringers	1,850,000	77.6	32.2	0.22	.048	1.55	79.2	147,000,000
Bottom Skin	1,980,000	.100	7.66	3.06	9.36	71.7	71.8	142,000,000
Total							I_g = 235.9	457,000,000

$$EI_g = 457,000,000 \text{ lb-in.}^2 \text{ per 4-ft width}$$

B.

FIGURE A1-2 NEUTRAL AXIS FOR DEFLECTION AND (EI_g)
(Calculations Examples)[1(p.8)]

(FF or fixed ends beam assumption), based on cross-panel top skin deflection behavior, as follows:

$$p_t = 384\ EI\ /\ [C(\ell'')^3] + DL$$

where: p_t = allowable TL - top skin deflection (psi)

C = factor for max. allowable deflection*
(often 360 floors, 240 roofs, LL only)

E is for top skin[2(p.17, no 10% added)] (psi)

I is for stress applied perpendicular to stringers and face grain[2(p.16,col.9)]; table's in.^{4}/ft values must be changed to in.^{4}/in.

ℓ'' = clear distance between stringers (Step 2 and Fig 1A) (should be uniform; if not, use longest value)(in.)

Mid-span cross-panel deflection, of course, then equals ℓ''/C.

7. Calculate N.A. for bending. Effective width of skins (as "flanges" to each stringer) is b/2 on each side of stringer, plus the width of the stringer. Get b from Step 2. Make sketch showing effective widths with each stringer, of both top and bottom skins. Calculate N.A. location, using bottom of panel as reference line for moment arms y; see example, Figure 3A; E values are used plus percentages, as in Step 3. Recall that $A_{//}$ tabular values are in in.^{2}/ft width and must be corrected for effective width of skins (versus total width used in Step 3), as must I_o skin values; moment arms for skins and stringers are the same as in Step 3. NOTE: Non-Stress-Graded stringers are omitted in the calculations of this Step (i.e., valued at zero), even though in Steps 3 and 4 they would be included.

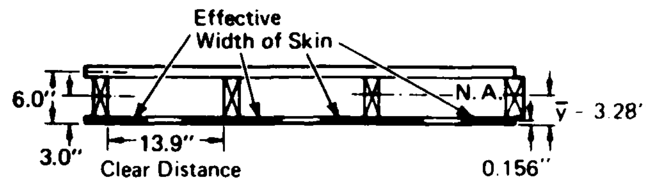
8. Calculate (EI_n) for bending. Use all data from Step 7, plus using I_o for each skin as flanges (from Step 4, but correcting I_o values from full panel width to "effective widths" of Step 7), again correcting for tabular units of in.^{4}/ft width, as necessary; use I_o values for stringers, as in Step 4 (omit Non-Stress-Graded stringers, though, as in Step 7). See example calculations, Figure 3B.

---

* If C is based on TL, then p_t will be directly in TL units (psi) without adding the DL term in the equation.[1(p.9)]

Effective width of top skin = 48″

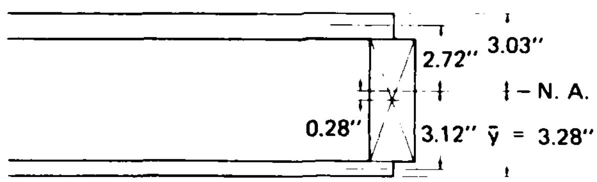
Effective width of bottom skin = 48″ -3(13.9 − 12) = 42.3″



Item	E	A∥	A∥E	y	A∥Ey
Top Skin	1,980,000	10.9	21,600,000	6.00	129,600,000
Stringers	1,850,000	32.2	59,600,000	3.00	178,800,000
Bottom Skin	1,980,000	$\frac{42.3}{12} \times 1.914 = 6.75$	13,400,000	0.156	2,090,000
			94,600,000		310,490,000

$$\bar{y} = \frac{\Sigma A \parallel Ey}{\Sigma A \parallel E} = \frac{310{,}490{,}000}{94{,}600{,}000} = 3.28''$$

A.



Item	E	I_o	A∥	d	d^2	A∥d^2	I_o + A∥d^2	E(I_o + A∥d^2)
Top Skin	1,980,000	0.564	10.9	2.72	7.40	80.7	81.3	161,000,000
Stringers	1,850,000	77.6	32.2	0.28	0.078	2.51	80.1	148,000,000
Bottom Skin	1,980,000	0.088	6.75	3.12	9.73	65.7	65.8	130,000,000
							I_n = 227.2	439,000,000

$$EI_n = 439{,}000{,}000 \text{ lb-in.}^2 \text{ per 4-ft width}$$

B.

FIGURE A1-3 NEUTRAL AXIS FOR BENDING MOMENT AND (EI_n) (Calculations Examples)[1(p.10,11)]

9. Determine top skin allowable compressive stress. Obtain F_c.[2(p.17)] Correct F_c using ratio of clear distance between stringers ℓ'' (Step 6) to ℓ (Step 5), as follows: for ratio [illegible], use 100%; for ratio [illegible] 1.0 up to [illegible] (see parenthetic comment in Step 2), use [illegible]%; and for ratios between [illegible] and [illegible], vary percentage correction linearly between 100% and [illegible]%.[1(p.11)]*

10. Determine bottom skin allowable tensile stress. Obtain F_t.[2(p.17)] Correct F_t in same manner as Step 9 (for F_c), using b_t from Step 2.[1(p.11)]

11. Calculate allowable load (TL) - bending:

$$p_b = (8F / (c\ell'\ell^2))((EI_n) / E)$$

where, p_b = allowable load (TL) - bending (psi)

$F = F_c$ or F_t from Steps 9 and 10, as appropriate (psi)

(EI_n) from Step 8 (lb-in.^{2})

E for skin under check, top or bottom (as in Step 3, including percentage increase)(psi)

c = distance from N.A. for <u>bending</u> (Step 7) to extreme fibre (of skin under check, top or bottom)(in.)

ℓ and ℓ' are same as in Step 5 (in.)

Check p_b for both top (p_{bt}) and bottom (p_{bb}) skins, then use smaller value as the applicable p_b.

12. Calculate allowable load (TL) - rolling shear:

It is generally sufficient to check rolling shear only in the thicker skin (it usually has the larger Q_s of the two skins, which leads to a smaller allowable load).[1(p.13)] The skin's <u>critical plane</u> for checking (in panels with face plies parallel to stringers, a fundamental limitation in the overall procedure herein) is along the glued plane on the inner side of the inside face ply of the panel; see Figure 4A.

Find area A (in.^{2}) for parallel-grain plies outside the critical plane (note that tabular values are for 48-in. wide panels, so must be corrected proportionately for other panel widths ℓ'), Figure 4B, 2nd or

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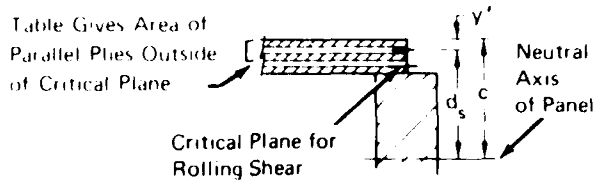
* Reference 1, p.11, figure erroneously shows 67.5% instead of correct value of 66.7% (as shown in text example and in other sources).



A.

A and y' for Computing Q_s *

Plywood Thickness (t)	STRUCTURAL I Grades, or any Group 4 Panel				All Other Panels			
	Face Grain to Stringers		Face Grain ⊥ to Stringers		Face Grain to Stringers		Face Grain ⊥ to Stringers	
	Area (in.²)	y' (in.)	Area (in.²)	y' (in.)	Area (in.²)	y' (in.)	Area (in.²)	y' (in.)
Unsanded Panels								
5/16	4.75	0.0495	4.75	0.149	3.83	0.0479	2.64	0.149
3/8	4.45	0.0464	5.75	0.180	3.73	0.0467	3.19	0.180
1/2	5.81	0.0606	7.75	0.242	6.25	0.131	4.31	0.242
5/8	9.24	0.176	9.75	0.305	7.18	0.140	5.42	0.305
3/4	7.34	0.0765	11.8	0.367	8.16	0.208	6.53	0.367
Sanded Panels								
1/4	3.36	0.0350	4.91	0.121	3.36	0.0350	2.73	0.121
3/8	3.36	0.0350	8.51	0.184	3.36	0.0350	4.73	0.184
1/2	3.89	0.0406	9.23	0.246	3.89	0.0406	5.13	0.246
5/8	4.56	0.0475	11.7	0.309	4.56	0.0475	6.51	0.309
3/4	12.0	0.277	15.1	0.371	8.18	0.233	8.42	0.371
Touch Sanded Panels								
1/2	4.56	0.0475	8.35	0.224	4.56	0.0475	4.64	0.224
19/32	7.93	0.174	11.6	0.276	5.90	0.139	6.44	0.276
5/8	8.21	0.185	12.3	0.288	6.06	0.148	6.86	0.288
23/32	6.06	0.0632	14.5	0.344	8.14	0.213	8.06	0.344
3/4	6.06	0.0632	15.3	0.356	8.37	0.225	8.50	0.356
2-4-1					12.7	0.359	16.5	0.547

* Area based on 48" wide panel. For other widths, use a proportionate area.

B.

FIGURE A1-4 ROLLING SHEAR CRITICAL PLANE AND Q_s 1(p.14)

6th column. Calculate distance d_s (in.) from N.A. (for <u>deflection</u>, Step 3) to centroid of A, using moment arm $d_s = c - y'$ (all in. units), where c is distance from N.A. to extreme fibre and y' can be taken from table, Figure 4B. Then calculate the statical moment Q_s (in.^{3}):

$$Q_s = A\ d_s$$

Calculate $\Sigma F_s t$ (psi-in., or lb/in.), the sum of the glueline widths over each stringer, each multiplied by its applicable allowable rolling shear stress F_s (Reference 2, page 17) but with a 50% reduction applied to outer stringer(s) whose clear distance to a panel edge is less than half the clear distance between stringers.[1(p.15)]

Calculate allowable load (TL) - rolling shear (p_s, psi):

$$p_s = (2(\Sigma F_s t) / (\ell\ell' Q_s))\ \ ((EI_g) / E)$$

where: $(\Sigma F_s t)$(lb./in.), ℓ and ℓ'(in.), and Q_s(in.^{3}) are defined above

(EI_g) from Step 4 (lb-in.^{2})

E for skin under check, usually thicker one (tabular value plus 10% if taken from Ref. 2, p. 17)

13. Calculate allowable load (TL) - horizontal shear:

Calculate statical moment Q_v of all parallel-grain plies and stringers in full panel width ℓ', working <u>either</u> above <u>or</u> below* the N.A. for deflection (Step 3 and Figure 2 can provide numerical data for these Q_v = A d calculations, as an example of course):

$$Q_v = Q_{stringers} + Q_{skin} \left[E_{skin} / E_{stringers} \right]$$

where: Q_v is defined above (in.^{3})

$Q_{stringers}$ = x-sec. area of all stringer portions <u>either</u> above <u>or</u> below N.A. (depending on chosen approach)* times its centroidal distance from deflection N.A. (as moment arm) (in.^{3})

$Q_{skin} = A_{//}$ for chosen skin × moment arm (in.^{3})[2(p.16, col. 4 for $A_{//}$)]

E's as before (Step 3, including percentage increases)(psi)

---

* Calculations "below" are easier, if deflection N.A. calculations (Step 3) were made as stated, i.e., using bottom surface of panel as reference plane.

Calculate:

$$p_v = (2\ F_v t\ /\ (\ell\ell' Q_v))\ \ ((EI_g)\ /\ E_{st})$$

where: p_v = allowable load (TL) - horizontal shear (psi)

F_v = allowable stress in stringer horizontal shear (psi)[3(Table 1)]

t = sum of stringer widths (including side projecting portions, Figures 1A and 2)(in.)

(EI_g) from Step 4 (lb-in.^{2})

E_{st} for stringers, as in Step 3 including percentage increase (psi)

ℓ, ℓ', and Q_v as above (in., in., and in.^{3})

14. Calculate required end bearing length:

The preceding steps that have led to allowable load (TL) under various criteria have used ℓ = clear span (in.)(Steps 5,6,11,12 and 13), but for end bearing, the full length of the panel will be greater than ℓ, sufficiently to provide for the allowable load (TL) in end bearing. Further, properly installed headers will have to be capable of spreading the end bearing load across the full panel width of the (thin) bottom skin; thus continuous headers crossing (nail-glued or pressure-glued) the stringer ends, and within the cover of both top and bottom skins, are recommended (see Reference 1, page facing page 1, top sketch, far end, for example).

The following approach to calculating ℓ_e (required plywood end bearing length at each end) considers adoption of the continuous-headers recommendation just above, but may be also used, perhaps with less confidence in ultimate strength behavior, for blocking-type headers (see same Reference 1 sketch, near end, for an example).

Let: ℓ_e = required plywood end bearing length at each end of panel (in.)

ℓ = clear span of panel, as before (in.)

ℓ' = full panel width (skins only)(note: entire panel area, including end bearing lengths, are assumed to be under a uniform loading)(in.)

p_m = smallest of the calculated allowable loads (TL), from Steps 5, 6, 11, 12 and 13 (psi)

$F_{c\perp}$ = allowable bearing stress on plywood face, for load perpendicular to plane of outer ply actually in bearing (psi)[2(p.17)]

Then: applied load must be less than or equal to resisting capacity:

$$p_m \ell \ell' \le 2\, \ell' \ell_e F_{c\perp}$$

or ℓ_e (min. at each end of panel) $= p_m \ell \,/\, (2\, F_{c\perp})$

It is recommended that ℓ_e be at least 1.5 in. (38 mm).

The bearing length of each stringer end (at least 1.5 or 2 inches) (38 or 51 mm) should be sufficient to handle the unit blast load on the plywood panel multiplied by the maximum c-c spacing of stringers and divided by the stringer width, all in accordance with Appendix B (especially Figure 6-12, which may be extended as needed based on last "bullet" paragraphs on page 6-111). See also Figures 9 later herein.

15. Glued plywood end joints (across face grain):[2(p.25,Sec.5.6)]

15A. Scarf joints: Sketches of end-of-grain joints are available.[4(p.9-11)] Scarf joints are made by bevelling across the plywood end edges (i.e., perpendicular to stringers and face plies of top and bottom skins), then joining the bevelled ends with an appropriate adhesive.

For the tension skin: 1 in 8 or flatter bevels transmit 100% of full allowable stress; 1 in 5 transmit 75%; use linear proportioning between these two bevels; and steeper than 1 in 5 are not to be used.[2(p.25)]

For compression skin: 1 in 5 or flatter bevels transmit 100% of allowable stress; steeper than 1 in 5 are not to be used.[2(p.26)]

(Note: Finger joints are too complicated to form and otherwise unsuitable for further consideration herein.)

15B. Splice-plate design (butt joints): While scarf joints are the recommended technique, this design section is presented for use if needed.[1(p.12,Sec.2.5.6)]

For a splice-plate illustration, see Figure 1[(2,p.4)] or top sketch of page facing page 1 of reference 1.

Splice-plates are to: be 1/4 in. clear of stringers at both plate ends; have skin face grain perpendicular to splice; be of grade and species group equal to the plywood spliced; and be no thinner than the skin being spliced. Tension skins with splice-plates are capable of transmitting 100% of maximum allowable stress.[2(p.26,table)] If the splice-plate is shorter than required for use of an allowable stress in the referenced table, the allowable stress is to be reduced proportionately.

Calculate splice plate allowable load (TL) - tension:

$$p_p = (8\ F\ /\ (c\ell'\ell^2))\ \ ((EI_g)\ /\ E)$$

where: p_p = allowable load (TL) on tension splice at point of max. moment (psi)

F = allowable splice-plate stress × proportion of panel width actually spliced[2(p.26,table)]

c = distance from deflection neutral axis to extreme bottom (tension) fibre (in.)

c and (EI_g) are as in Figure 2A ($\bar{y}$) and Step 4, respectively (in. and lb-in.^{2})

E is for tension skin, as used in Step 3 (with the percentage increase)(psi)

ℓ and ℓ' are as before (in.)

Splice plate allowable load (TL) - compression: These plates can be approved by inspection, for 100% transmittal of allowable stress, subject to cited references.[2(Sec.5.6.1.2 and 5.6.2.2)]

Design Stresses - Blast Protection Use versus Normal Use. The design procedure detailed above is that for normal, day-to-day uses, for which allowable stresses are prescribed.[2(p.17),3] Such allowable stresses are totally inappropriate for one-time blast loadings, with their extremely short (essentially zero) rise-times and short durations (1 or 2 seconds in our range of interest, even for megaton weapons), inappropriate in that they result in seriously underestimating the ultimate strength of structural members under blast loadings. The reader is referred to Appendix B1 herein, especially the introductory section and the "Design Procedure" section; within the latter, specific attention is invited to its introductory section and design steps 1 through 4. Such referenced reading covers the very basic structural dynamics, bilinear blast resistance, ductility ratio μ, etc., as well as the increased stresses used in blast-resistant design: for wood beams, the increases are four times for F_b and F_v (extreme fibre stress in bending and horizontal shear stress, respectively) and no increase in $F_{c\perp}$ (compression stress perpendicular to grain, or bearing stress). Authorities are cited.

An examination of literature helpfully furnished by the U.S. Forest Products Laboratory, Madison, Wisconsin, indicated the following: Tests on plywood stressed-skin panels (PSSPs) to destruction were few in the literature furnished, being restricted to tests on PSSPs with narrow, plywood stringers where all (predictably) failed along the stringer gluelines; the allowable stress increase of 100% for impact loads[2(Sec.3.3.1.1)] seems to be well supported by a test report[5] in terms of both short duration loads and fast rate of loading, for both wood and wood-based materials (including plywood).

If one considers that "allowable" stresses in most cases (and materials types) are based on a factor of safety of about two, we can then arrive at a factor of four (including the 100% for impact) for ultimate strength under short, rapidly applied loads - a factor of four for certain stresses, at least. These stresses might include, for PSSPs:[2(p.17)] F_b, F_t, F_c, F_v and F_s, but not $F_{c\perp}$.

Pending receipt of better information based on sorely needed tests, these dynamic stress increases were tentatively adopted for use herein; test data found were for static loadings, or for loads within severe limits on deflection, or for loadings far short of failure/collapse (as used in typical air blast loadings and design technology).[5,6]

For a value of the ductility ratio μ, however (see App. B1, Design Step 3), a value of two was similarly and tentatively adopted for PSSPs, with a value of three tentatively continued for wood beams,* again hoping to obtain appropriate test information in the early future. For a

* See Appendix B1 following.

dynamic load simplified to a step pulse (zero rise-time to a constant loading of infinite duration), the relationship is

$$p_{dm} = p_m \, (1 - 1/(2\mu)) = p_m(3/4), \text{ for } \mu = 2$$

Typical Designs of PSSPs. In order to handle the many sets of design computations required in producing a reasonably adequate catalog of pre-designs, a computer program was prepared (in Dartmouth BASIC), following the above 15-step design procedure (Step 15 on design of plywood end joints was not included in the design output, although it is included in the computer program). A listing of the computer program is used, but as revised for use there, is shown in Appendix A2.

The pre-designs* covered clear span ranges from 24" to 96" for lighter panels and from 24" to 144" for heavier. Stringers included 2x4s, 2x6s and 2x8s of both relatively low and high strengths, thereby covering a considerable range of lumber species among those readily available in local lumberyards. Several plywood types/species/grades were examined, with complete pre-designs using two types/grades throughout; this was coupled with use of face ply species groups #1 and #3, except that #3 was not used for the 1-1/8" plywood because of unavailability.

The pre-designs of Table 1 are limited to two plywoods: Underlayment Interior (APA) in face ply group species #1 and #3, for 1/2", 5/8", and 3/4" thicknesses; and 2.4.1 Sturd-I-Floor Interior (APA), which is only made in #1, for the 1-1/8" thickness. These plywoods have high availability in local lumberyards in the indicated thicknesses.

Also having similar availability are three other plywood grades: Underlayment Exterior (APA), C-D Interior (APA) and C-C Exterior (APA). Pre-designs were prepared using these three plywoods in sufficient number to show that Table 1 may also be used for them with insignificant errors (all on the conservative side).

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* All included panel dead load (DL), which was less than 0.1 psi in all cases, thus p_{dm} values are appropriate for laterally loaded panels used horizontally or vertically.

Table A1-1A PSSP DESIGNS FOR LOWER STRENGTH STRINGERS (F_v = 280 psi)*
(Beams)

Panel Width: 48 in.

TOP SKIN		BOT. SKIN		STRNGRS		REQD. BEAR. EACH END		(FREE FIELD, SIDE-ON, LONG DURATION) PEAK AIR BLAST OVERPRESSURE psi VS. CLEAR SPAN																				
								Clear Span, ft																				
Nom. Th. in.	Face Ply Grp	Nom. Th. in.	Face Ply Grp	Nom. Size in.	No.	Ply. Bot. Skin in.	Str-ngrs in.	2	2½	3	3½	4	4½	5	5½	6	6½	7	7½	8	8½	9	9½	10	10½	11	11½	12
1/2	#1	1/2	#1	2X4	4	1.5	4.5	7	6	5																		
					5	1.5	4.0	9	7	6	5																	
					7	1.5	3.5	11	9	8	6	6	5	5														
					9	1.5	3.5	14	11	9	8	7	6	6	5	5												
1/2	#3	1/2	#3	2X4	4	1.5	4.5	7	5																			
					5	1.5	4.0	8	6	5	5																	
					7	1.5	3.5	10	8	7	6	5	5															
					9	1.5	3.0	12	10	8	7	6	6	5	5													
5/8	#1	1/2	#1	2X4	4	1.5	5.0	7	6	5																		
					5	1.5	4.5	9	7	6	5																	
					7	1.5	4.0	12	9	8	7	6	5	5														
					9	1.5	3.5	14	11	9	8	7	6	6	5	5												
5/8	#3	1/2	#3	2X4	4	1.5	4.5	7	5	5																		
					5	1.5	4.0	8	6	5	5																	
					7	1.5	3.5	11	8	7	6	5	5															
					9	1.5	3.0	13	10	8	7	6	6	5	5													
3/4	#1	1/2	#1	2X4	4	1.5	5.0	8	6	5																		
					5	1.5	4.5	9	7	6	5	5																
					7	1.5	4.0	12	10	8	7	6	5	5														
					9	1.5	3.5	15	12	10	8	7	7	6	5	5	5											
3/4	#3	1/2	#3	2X4	4	1.5	4.5	7	6	5																		
					5	1.5	4.0	8	7	6	5																	
					7	1.5	3.5	11	9	7	6	6	5															
					9	1.5	3.5	13	11	9	8	7	6	5	5													
3/4	#1	3/4	#1	2X4	4	1.5	5.0	8	6	5	5																	
					5	1.5	4.5	10	8	6	5	5																
					7	1.5	4.0	13	10	8	7	6	6	5	5													
					9	1.5	4.0	15	12	10	9	8	7	6	6	5	5											
3/4	#3	3/4	#3	2X4	4	1.5	5.0	7	6	5																		
					5	1.5	4.5	9	7	6	5																	
					7	1.5	4.0	12	9	8	7	6	5	5														
					9	1.5	3.5	14	11	9	8	7	6	6	5	5												
1-1/8	#1	1/2	#1	2X4	4	1.5	5.0	7	6	5																		
					5	1.5	4.5	9	8	6	5	5																
					7	1.5	4.5	13	11	9	8	7	6	5	5													
					9	1.5	4.0	16	13	11	9	8	7	7	6	5	5	5										
1-1/8	#1	1/2	#3	2X4	4	1.5	4.5	7	6	5																		
					5	1.5	4.5	9	7	6	5	5																
					7	1.5	4.5	13	11	9	8	7	6	5	5													
					9	1.5	4.0	16	13	11	9	8	7	6	6	5	5	5										

Table A1-1A (continued)

1-1/8	#1	3/4	#1	2X4	4	1.5	5.0	8	6	5															
					5	1.5	5.0	10	8	7	6	5													
					7	1.5	4.5	14	11	9	8	7	6	6	5	5									
					9	1.5	4.0	17	14	11	10	9	8	7	6	6	5	5	5						
1-1/8	#1	3/4	#3	2X4	4	1.5	5.0	7	6	5															
					5	1.5	4.5	10	8	6	5	5													
					7	1.5	4.5	14	11	9	8	7	6	5	5	5									
					9	1.5	4.0	17	13	11	9	8	7	7	6	6	5	5							
1-1/8	#1	1-1/8	#1	2X4	4	1.5	5.5	9	7	6	5														
					5	1.5	5.0	11	9	7	6	5	5												
					7	1.5	5.0	15	12	10	9	7	7	6	5	5	5								
					9	1.5	4.5	18	15	12	10	9	8	7	7	6	6	5	5	5					
1/2	#1	1/2	#1	2X6	4	1.5	7.5	11	9	8	7	6	5	5											
					5	1.5	6.5	14	11	9	8	7	6	6	5	5									
					7	1.5	6.0	19	15	12	11	9	8	7	7	6	6	5	5	5					
					9	1.5	5.5	23	19	15	13	12	10	9	8	8	7	7	6	6	5	5	5	5	
1/2	#3	1/2	#3	2X6	4	1.5	7.0	11	9	7	6	5	5												
					5	1.5	6.5	14	11	9	8	7	6	5	5	5									
					7	1.5	5.5	18	14	12	10	9	8	7	6	6	5	5	5						
					9	2.0	5.5	22	17	15	12	11	10	9	8	7	7	6	6	5	5	5			
5/8	#1	1/2	#1	2X6	4	1.5	7.5	12	9	8	7	6	5	5											
					5	1.5	7.0	14	11	9	8	7	6	6	5	5									
					7	1.5	6.0	19	15	13	11	9	8	8	7	6	6	5	5	5					
					9	1.5	5.5	23	19	16	13	12	10	9	9	8	7	7	6	6	6	5	5	5	
5/8	#3	1/2	#3	2X6	4	1.5	7.0	11	9	7	6	6	5												
					5	1.5	6.5	14	11	9	8	7	6	6	5	5									
					7	1.5	6.0	18	14	12	10	9	8	7	6	6	5		5						
					9	2.0	5.5	22	18	15	13	11	10	9	8	7	7	6	6	6	5	5			
3/4	#1	1/2	#1	2X6	4	1.5	7.5	12	10	8	7	6	5	5											
					5	1.5	7.0	15	12	10	8	7	6	6	5	5									
					7	1.5	6.5	20	16	13	11	10	9	8	7	7	6	6	5	5	5				
					9	1.5	6.0	24	20	16	14	12	11	10	9	8	8	7	7	6	6	5	5	5	5
3/4	#3	1/2	#3	2X6	4	1.5	7.5	11	9	8	7	6	5	5											
					5	1.5	7.0	14	11	10	8	7	6	6	5	5									
					7	1.5	6.0	19	15	12	11	9	8	7	7	6	6	5	5	5					
					9	2.0	5.5	23	18	15	13	11	10	9	8	8	7	7	6	6	5	5			
3/4	#1	3/4	#1	2X6	4	1.5	8.0	12	10	8	7	6	5	5											
					5	1.5	7.0	15	12	10	8	7	7	6	5	5	5								
					7	1.5	6.5	20	16	13	11	10	9	8	7	7	6	6	5	5	5				
					9	1.5	6.0	25	20	16	14	12	11	10	9	8	8	7	7	6	6	5	5	5	5
3/4	#3	3/4	#3	2X6	4	1.5	7.5	12	9	8	7	6	5	5											
					5	1.5	7.0	14	12	10	8	7	6	6	5	5									
					7	1.5	6.0	19	15	13	11	9	8	8	7	6	6	5	5	5					
					9	2.0	5.5	23	19	15	13	12	10	9	8	8	7	7	6	6	5	5	5		

Table A1-1A PSSP DESIGNS FOR LOWER STRENGTH STRINGERS (F_v = 280 psi)* (concluded)

Panel Width: 48 in.

TOP SKIN Nom. Th. in.	TOP SKIN Face Ply Grp	BOT. SKIN Nom. Th. in.	BOT. SKIN Face Ply Grp	STRNGRS Nom. Size in.	STRNGRS No.	REQD. BEAR. EACH END Ply. Bot. Skin in.	REQD. BEAR. EACH END Str-ngrs in.	2	2½	3	3½	4	4½	5	5½	6	6½	7	7½	8	8½	9	9½	10	10½	11	11½	12
								(FREE FIELD, SIDE-ON, LONG DURATION) PEAK AIR BLAST OVERPRESSURE psi VS. CLEAR SPAN — Clear Span, ft																				
1-1/8	#1	1/2	#1	2X6	4	1.5	7.5	12	9	8	7	6	5	5														
					5	1.5	7.0	15	12	10	8	7	7	6	5	5	5											
					7	1.5	7.0	21	17	14	12	11	10	9	8	7	7	6	6	5	5	5	5					
					9	1.5	6.5	27	21	18	15	13	12	11	10	9	8	8	7	7	6	6	6	5	5	5	5	
1-1/8	#1	1/2	#3	2X6	4	1.5	7.0	11	9	7	6	5	5															
					5	1.5	7.0	15	12	10	8	7	6	6	5	5												
					7	2.0	7.0	22	17	15	12	11	10	9	8	7	7	6	6	5	5	5	5					
					9	2.0	6.5	27	22	18	15	14	12	11	10	9	8	8	7	7	6	6	6	5	5	5		
1-1/8	#1	3/4	#1	2X6	4	1.5	8.0	12	10	8	7	6	5	5														
					5	1.5	7.5	15	12	10	9	8	7	6	6	5	5											
					7	1.5	7.0	22	17	14	12	11	10	9	8	7	7	6	6	5	5	5	5					
					9	1.5	6.5	27	22	18	15	13	12	11	10	9	8	8	7	7	6	6	6	5	5	5	5	
1-1/8	#1	3/4	#3	2X6	4	1.5	7.5	11	9	8	7	6	5	5														
					5	1.5	7.0	15	12	10	9	7	7	6	5	5	5											
					7	2.0	7.0	22	18	15	13	11	10	9	8	7	7	6	6	5	5	5	5					
					9	2.0	6.5	27	22	18	16	14	12	11	10	9	8	8	7	7	6	6	6	5	5	5	5	
1-1/8	#1	1-1/8	#1	2X6	4	1.5	8.5	13	11	9	8	7	6	5	5													
					5	1.5	8.0	16	13	11	9	8	7	6	6	5	5	5										
					7	1.5	7.0	22	18	15	13	11	10	9	8	7	7	6	6	5	5	5	5					
					9	1.5	6.5	27	22	18	16	14	12	11	10	9	8	8	7	7	6	6	6	5	5	5	5	5
1-1/8	#1	1/2	#1	2X8	4	1.5	10.0	16	12	10	9	8	7	6	6	5	5											
					5	1.5	10.0	20	16	14	12	10	9	8	7	7	6	6	5	5	5	5						
					7	1.5	9.5	30	24	20	17	15	13	12	11	10	9	9	8	8	7	7	6	6	6	5	5	5
					9	2.0	9.0	38	30	25	22	19	17	15	14	13	12	11	10	10	9	8	8	8	7	7	7	6
1-1/8	#1	1/2	#3	2X8	4	1.5	9.5	15	12	10	8	7	7	6	5	5	5											
					5	1.5	10.0	20	16	13	12	10	9	8	7	7	6	6	5	5	5							
					7	2.5	10.0	31	25	21	18	16	14	13	11	10	10	9	8	8	7	7	7	6	6	6	5	5
					9	3.0	9.5	39	31	26	22	20	17	16	14	13	12	11	10	10	9	9	8	8	7	7	6	6
1-1/8	#1	3/4	#1	2X8	4	1.5	10.5	16	13	11	9	8	7	7	6	5	5	5										
					5	1.5	10.0	21	17	14	12	10	9	8	8	7	6	6	6	5	5	5						
					7	1.5	9.5	30	24	20	17	15	13	12	11	10	9	9	8	7	7	7	6	6	6	5	5	5
					9	2.0	9.0	38	30	25	21	19	17	15	14	13	12	11	10	9	9	8	8	8	7	7	7	6
1-1/8	#1	3/4	#3	2X8	4	1.5	10.0	15	12	10	9	8	7	6	6	5	5											
					5	1.5	10.0	21	16	14	12	10	9	8	7	7	6	6	5	5	5	5						
					7	2.5	10.0	31	25	21	18	15	14	12	11	10	10	9	8	8	7	7	7	6	6	6	5	5
					9	3.0	9.5	39	31	26	22	19	17	16	14	13	12	11	10	10	9	9	8	8	7	7	7	6
1-1/8	#1	1-1/8	#1	2X8	4	1.5	11.0	17	14	12	10	9	8	7	6	6	5	5	5									
					5	1.5	10.5	21	17	14	12	11	9	9	8	7	7	6	6	5	5	5						
					7	1.5	9.5	29	23	20	17	15	13	12	11	10	9	8	8	7	7	7	6	6	6	5	5	5
					9	2.0	9.0	37	30	25	21	19	16	15	13	12	11	11	10	9	9	8	8	7	7	7	6	6

Table A1-1B PSSP DESIGNS FOR HIGHER STRENGTH STRINGERS (F_v = 380 psi)*

Panel Width: 48 in.

TOP SKIN		BOT. SKIN		STRNGRS		REQD. BEAR. EACH END		(FREE FIELD, SIDE-ON, LONG DURATION) PEAK AIR BLAST OVERPRESSURE psi VS. CLEAR SPAN																				
								Clear Span, ft.																				
Nom. Th. in.	Face Ply Grp	Nom. Th. in.	Face Ply Grp	Nom. Size in.	No.	Ply. Bot. Skin in.	Str-ngrs in.	2	2½	3	3½	4	4½	5	5½	6	6½	7	7½	8	8½	9	9½	10	10½	11	11½	12
1/2	#1	1/2	#1	2X4	4	1.5	3.5	9	7	6	5	5																
					5	1.5	3.5	11	9	7	6	5	5															
					7	1.5	3.0	14	11	9	8	7	6	6	5	5												
					9	1.5	2.5	17	14	11	10	8	8	7	6	6	5	5	5									
1/2	#3	1/2	#3	2X4	4	1.5	3.5	8	7	6	5																	
					5	1.5	3.0	10	8	7	6	5																
					7	1.5	2.5	13	10	8	7	6	6	5	5													
					9	1.5	2.5	15	12	10	9	8	7	6	6	5	5											
5/8	#1	1/2	#1	2X4	4	1.5	3.5	9	7	6	5	5																
					5	1.5	3.5	11	9	7	6	5	5															
					7	1.5	3.0	14	11	10	8	7	6	6	5	5												
					9	1.5	2.5	17	14	11	10	9	8	7	6	6	5	5	5									
5/8	#3	1/2	#3	2X4	4	1.5	3.5	8	7	6	5																	
					5	1.5	3.0	10	8	7	6	5																
					7	1.5	2.5	13	10	9	7	6	6	5	5													
					9	1.5	2.5	15	12	10	9	8	7	6	6	5	5											
3/4	#1	1/2	#1	2X4	4	1.5	3.0	8	6	5	5																	
					5	1.5	3.0	10	8	7	6	5	5															
					7	1.5	3.0	15	12	10	9	8	7	6	5	5	5											
					9	1.5	3.0	18	14	12	10	9	8	7	7	6	6	5	5	5								
3/4	#3	1/2	#3	2X4	4	1.5	3.5	8	7	6	5																	
					5	1.5	3.0	10	8	7	6	5																
					7	1.5	3.0	13	11	9	8	7	6	5	5													
					9	1.5	2.5	16	13	11	9	8	7	6	6	5	5	5										
3/4	#1	3/4	#1	2X4	4	1.5	3.5	8	7	6	5																	
					5	1.5	3.5	11	9	7	6	5	5															
					7	1.5	3.0	16	13	11	9	8	7	6	6	5	5	5										
					9	1.5	3.0	19	15	13	11	10	8	8	7	6	6	5	5	5								
3/4	#3	3/4	#3	2X4	4	1.5	3.5	9	7	6	5	5																
					5	1.5	3.5	11	9	7	6	5	5															
					7	1.5	3.0	14	11	9	8	7	6	6	5	5												
					9	1.5	2.5	17	14	11	10	9	8	7	6	6	5	5										
1-1/8	#1	1/2	#1	2X4	4	1.5	3.0	7	6	5																		
					5	1.5	3.0	10	8	7	6	5	5															
					7	1.5	3.0	16	12	10	9	8	7	6	6	5	5											
					9	1.5	3.0	20	16	14	12	10	9	8	7	7	6	6	5	5	5	5						
1-1/8	#1	1/2	#3	2X4	4	1.5	3.0	7	6	5																		
					5	1.5	3.0	10	8	6	6	5																
					7	1.5	3.0	15	12	10	8	7	7	6	5	5	5											
					9	1.5	3.0	20	16	13	11	10	9	8	7	7	6	6	5	5	5							

Table A1-1B PSSP DESIGNS FOR HIGHER STRENGTH STRINGERS (F_V = 380 psi)* (continued)

Panel Width: 48 in.

TOP SKIN		BOT. SKIN		STRNGRS		REQ'D. BEAR. EACH END		(FREE FIELD, SIDE-ON, LONG DURATION) PEAK AIR BLAST OVERPRESSURE psi VS. CLEAR SPAN																				
								Clear Span, ft																				
Nom. Th. in.	Face Ply Grp	Nom. Th. in.	Face Ply Grp	Nom. Size in.	No.	Ply. Bot. Skin in.	Str-ngrs in.	2	2½	3	3½	4	4½	5	5½	6	6½	7	7½	8	8½	9	9½	10	10½	11	11½	12
1-1/8	#1	3/4	#1	2X4	4	1.5	3.0	8	6	5	5																	
					5	1.5	3.5	11	9	7	6	5	5															
					7	1.5	3.5	17	13	11	10	8	7	7	6	6	5	5										
					9	1.5	3.0	21	17	14	12	11	9	8	8	7	7	6	6	5	5	5						
1-1/8	#1	3/4	#3	2X4	4	1.5	3.0	7	6	5																		
					5	1.5	3.0	10	8	7	6	5	5															
					7	1.5	3.0	16	12	10	9	8	7	6	6	5	5											
					9	1.5	3.0	20	16	14	12	10	9	8	7	7	6	6	5	5	5							
1-1/8	#1	1-1/8	#1	X4	[illegible]	1.5	3.5	9	7	6	5																	
						1.5	3.5	12	10	8	7	6	5	5														
						1.5	4.0	19	15	12	11	9	8	7	7	6	6	5	5	5								
						1.5	3.5	23	18	15	13	11	10	9	8	8	7	7	6	6	5	5	5					
1/2	#1	1/2	#1	X6	4	1.5	6.0	15	12	10	8	7	6	6	5	5												
					5	1.5	5.5	18	14	12	10	9	8	7	6	6	5	5	5									
					7	1.5	4.5	24	19	16	13	12	10	9	9	8	7	7	6	6	6	5	5	5				
					9	1.5	4.5	29	23	19	17	15	13	12	11	10	9	8	8	7	7	6	6	6	6	5	5	
1/2	#3	1/2	#3	2X6	4	1.5	5.5	14	11	9	8	7	6	6	5	5												
					5	1.5	5.0	17	14	11	10	8	8	7	6	6	5	5	5									
					7	2.0	4.5	22	18	15	13	11	10	9	8	7	7	6	6	6	5	5	5					
					9	2.5	4.0	28	22	18	16	14	12	11	10	9	9	8	7	7	7	6	5	5				
5/8	#1	1/2	#1	2X6	4	1.5	6.0	15	12	10	8	7	7	6	5	5	5											
					5	1.5	5.5	18	14	12	10	9	8	7	7	6	6	5	5									
					7	1.5	5.0	24	19	16	14	12	11	10	9	8	7	7	6	6	6	5	5	5	5			
					9	1.5	4.5	30	24	20	17	15	13	12	11	10	9	8	8	7	7	7	6	6	5	5	5	
5/8	#3	1/2	#3	2X6	4	1.5	5.5	14	11	9	8	7	6	6	5	5												
					5	1.5	5.0	17	14	12	10	9	8	7	6	6	5	5	5									
					7	2.0	4.5	22	18	15	13	11	10	9	8	7	7	6	6	6	5	5	5					
					9	2.5	4.0	28	22	19	16	14	12	11	10	9	9	8	7	7	7	6	5	5				
3/4	#1	1/2	#1	2X6	4	1.5	5.5	14	11	9	8	7	6	5	5	5												
					5	1.5	5.0	17	14	12	10	9	8	7	6	6	5	5	5									
					7	1.5	5.0	25	20	17	14	12	11	10	9	8	8	7	7	6	6	6	5	5	5	5		
					9	1.5	4.5	31	25	20	18	15	14	12	11	10	9	9	8	8	7	7	6	6	6	5	5	
3/4	#3	1/2	#3	2X6	4	1.5	6.0	15	12	10	8	7	6	6	5	5												
					5	1.5	5.5	18	15	12	10	9	8	7	7	6	6	5	5	5								
					7	2.0	4.5	23	19	16	13	12	10	9	8	8	7	7	6	6	5	5	5					
					9	2.5	4.5	29	23	19	16	14	13	12	10	10	9	8	8	7	7	6	5	5				
3/4	#1	3/4	#1	2X6	4	1.5	5.5	14	12	10	8	7	6	6	5	5												
					5	1.5	5.5	18	14	12	10	9	8	7	7	6	6	5	5									
					7	1.5	5.0	25	20	17	14	13	11	10	9	8	8	7	7	6	6	6	5	5	5	5		
					9	1.5	4.5	31	25	21	18	16	14	12	11	10	10	9	8	8	7	7	7	6	6	5	5	5

Table A1-1B (concluded)

3/4	#3	3/4	#3	2X6	4	1.5	6.0	15	12	10	8	7	7	6	5	5	5											
					5	1.5	5.5	18	15	12	10	9	8	7	7	6	6	5	5	5								
					7	2.0	4.5	24	19	16	13	12	10	9	9	8	7	7	6	6	6	5	5	5				
					9	2.5	4.5	29	23	19	17	15	13	12	11	10	9	8	8	7	7	6	6	5	5			
1-1/8	#1	1/2	#1	2X6	4	1.5	5.0	12	10	8	7	6	5	5														
					5	1.5	5.0	17	14	12	10	9	8	7	6	6	5	5	5									
					7	1.5	5.5	27	22	18	16	14	12	11	10	9	8	8	7	7	6	6	6	5	5	5	5	
					9	1.5	5.0	34	27	22	19	17	15	13	12	11	10	10	9	8	8	7	7	7	6	6	5	5
1-1/8	#1	1/2	#3	2X6	4	1.5	4.5	12	9	8	7	6	5	5														
					5	1.5	5.0	16	13	11	9	8	7	7	6	5	5	5										
					7	2.0	5.0	26	21	17	15	13	12	10	9	9	8	7	7	6	6	6	5	5	5	5	5	
					9	2.5	5.0	34	27	23	19	17	15	14	12	11	10	10	9	8	8	8	7	7	6	5	5	5
1-1/8	#1	3/4	#1	2X6	4	1.5	5.0	13	10	9	7	6	6	5	5													
					5	1.5	5.5	18	14	12	10	9	8	7	6	6	5	5	5									
					7	1.5	5.5	27	22	18	16	14	12	11	10	9	8	8	7	7	6	6	6	5	5	5	5	5
					9	1.5	5.0	34	27	23	19	17	15	14	12	11	10	10	9	8	8	8	7	7	6	6	6	5
1-1/8	#1	3/4	#3	2X6	4	1.5	5.0	12	10	8	7	6	5	5														
					5	1.5	5.0	17	14	11	10	9	8	7	6	6	5	5	5									
					7	2.0	5.5	27	22	18	15	13	12	11	10	9	8	8	7	7	6	6	6	5	5	5	5	
					9	2.5	5.0	34	27	23	19	17	15	14	12	11	10	10	9	9	8	8	7	7	6	6	5	5
1-1/8	#1	1-1/8	#1	2X6	4	1.5	5.5	14	11	9	8	7	6	6	5	5												
					5	1.5	5.5	19	15	12	11	9	8	7	7	6	6	5	5	5								
					7	1.5	5.5	28	22	19	16	14	12	11	10	9	9	8	7	7	7	6	6	6	5	5	5	5
					9	2.0	5.0	34	28	23	20	17	15	14	13	11	11	10	9	9	8	8	7	7	7	6	6	6
1-1/8	#1	1/2	#1	2X8	4	1.5	6.5	17	14	11	10	8	8	7	6	6	5	5	5									
					5	1.5	7.0	24	19	16	14	12	11	10	9	8	7	7	6	6	6	5	5	5	5			
					7	2.0	7.5	39	31	26	22	20	17	16	14	13	12	11	10	10	9	9	8	8	7	7	7	7
					9	2.5	7.5	49	39	33	28	25	22	20	18	16	15	14	13	12	12	11	10	10	9	9	8	[illegible]
1-1/8	#1	1/2	#3	2X8	4	1.5	6.5	16	13	11	9	8	7	6	6	5	5	5										
					5	2.0	7.0	23	19	15	13	12	10	9	8	8	7	7	6	6	5	5	5	5				
					7	3.0	7.5	38	30	25	21	19	17	15	14	13	12	11	10	9	9	8	8	8	7	7	7	6
					9	4.0	7.5	51	40	34	29	25	22	20	18	17	16	14	13	13	12	11	11	10	9	8	8	7
1-1/8	#1	3/4	#1	2X8	4	1.5	7.0	18	14	12	10	9	8	7	6	6	5	5	5									
					5	1.5	7.5	25	20	16	14	12	11	10	9	8	8	7	7	6	6	5	5	5	5			
					7	2.0	7.5	39	31	26	22	19	17	16	14	13	12	11	10	10	9	9	8	8	7	7	7	6
					9	2.5	7.0	49	39	33	28	24	22	20	18	16	15	14	13	12	11	11	10	10	9	9	8	8
1-1/8	#1	3/4	#3	2X8	4	1.5	6.5	17	13	11	9	8	7	7	6	6	5	5										
					5	2.0	7.0	24	19	16	14	12	11	10	9	8	7	7	6	6	6	5	5	5	5			
					7	3.0	7.5	39	31	26	22	19	17	16	14	13	12	11	10	10	9	9	8	8	7	7	7	6
					9	4.0	7.5	50	40	33	29	25	22	20	18	17	15	14	13	13	12	11	11	10	9	9	8	7
1-1/8	#1	1-1/8	#1	2X8	4	1.5	7.5	19	15	13	11	10	9	8	7	6	6	5	5	5	5							
					5	1.5	7.5	26	20	17	15	13	11	10	9	9	8	7	7	6	6	6	5	5	5	5		
					7	2.0	7.5	38	31	26	22	19	17	15	14	13	12	11	10	10	9	9	8	8	7	7	7	6
					9	2.5	7.0	48	39	32	28	24	21	19	18	16	15	14	13	12	11	11	10	10	9	9	8	8

* For dynamic loads (blast) use herein, F_v values were four times the normal use values, as published in Reference 3's Supplement.

PSSP Upgrading Example - Closure

Try one of the PSSP designs as a horiz. closure over an aperture (such as a stairwell), in the floor over a candidate basement shelter.

Assume use of the last PSSP design on page A1-20: 3/4" #1 both skins, 9 - 2x6 stringers (higher strength). Assume clear span of 4 ft.

From Table A1-18: peak blast overpr. = 16 psi

Factors included in the blast design are: 2.0 for impact duration of loading; 2.0 for factor of safety; and 0.75 for $\mu = 2$. See page A1-12.

Basic (static) load strength of PSSP =

$$\frac{16 \text{ psi}}{2.0 \times 2.0 \times 0.75} = 5.33 \text{ psi}$$

Fallout shielding soil:

Assume 18" th. layer of 95 pcf soil.

Allow 15% added stress for load duration less than 2 mos.

Static load strength used up for fallout soil $= \frac{1.5 \times 95}{144} \times \frac{1}{1.15} =$ 0.86 psi

Static load strength remaining = 4.47 psi

Blast:

Use static load strength remaining (4.47), increased by the 3 factors taken out above:

∴ Peak blast overpressure strength:

$$4.47 \times 2.0 \times 2.0 \times 0.75 = \underline{13.41 \text{ psi}}$$

Fabrication

Fabrication of plywood stressed-skin panels (PSSPs) is concisely yet thoroughly described in a publication available upon request.[7] The publication emphasizes the need for adequate gluing in order to develop the composite action of plywood stressed-skins and the stringers. Results from mechanical-pressure gluing have been found to be generally superior to nail-gluing (latter, properly performed, is the basis for the design section herein, however); supplies needed for nailing may have to be estimated in advance, for which the following extract will be useful:[7(p.6)]

> "Nails shall be at least . . . 6d for 1/2" to 7/8" plywood, 8d for 1" to 1-1/8" plywood, . . . spaced not to exceed . . . 4" (along the framing members) for plywood 1/2" and thicker, using one line for lumber 2" thick or less, and two lines for lumber more than 2" and up to 4" thick (wide)."

Glue, recommended for use in accordance with the manufacturers' recommendations, should be one of the two following types: Interior, for use when the equilibrium moisture content of the materials used does not exceed 18%, may be casein type with a mold inhibitor, conforming with ASTM Specification D3024; Exterior, for higher moisture contents, conforming to ASTM Specification D2559.

Nailing without gluing simply does not exploit the strength of PSSPs and the capabilities of their materials - the nails can too easily yield along the grain of the stringers so that they are inadequate as a shear transfer mechanism. The sparse test data found clearly show concern with deflection, not flexural, behavior as the controlling criterion, thus ultimate strength is not considered.

In the absence of some kind of ultimate strength behavior tests, the author has no basis for a recommendation, even heavily qualified, on the relative strength of nailed-only to nail- or pressure-glued PSSPs.

Further Work

As mentioned in the section on "Design Stresses . . ." above, tests for ultimate strength (i.e., through to failure/collapse, recording full load-deflection history including time) under dynamic loadings, or even under static loadings if well into the plastic range, are badly needed as a better basis for design of PSSPs as blast closures. With such information, one might be, for example, justified in design procedure use of numerical integration of the equation of motion, instead of the less rigorous approach of using a step-pulse loading of infinite duration, as has been done in preparing the design procedure above. Further, the wood design stresses would be better known, of course, as would the composite behavior including the primary cause of each test PSSP's failure mode. Some tests have been completed at the Ballistics Research Laboratory, U. S. Army, Aberdeen Proving Grounds, Maryland 21005, and more are planned.

NOTATION

A total x-section area of all stringers

A x-section area of parallel-grain plies outside the critical plane for rolling shear

A total x-section area of all stringers and skins $A_{//}$ (beam-columns)

$A_{//}$ x-section area (finished) of plies // stringers, in each skin

$A_{\perp}$ x-section area (finished) of plies $\perp$ stringers, in each skin

b, b_b, b_t basic stringer spacing; subscripts are for bottom and top skins, respectively

C factor for maximum allowable deflection (usually based on LL only)

c distance from neutral axis (for deflection or bending, as locally defined) to extreme fibre (of skin under check)(see $\bar{y}$)

d moment arms for various x-sectional areas (subscripted A's), used in I_g and I_n calculations

d_s $= c - y'$

E modulus of elasticity

E_{st} E of stringers

(EI_g) panel parameter, calculated using neutral axis for deflection

(EI_n) panel parameter, calculated using neutral axis for bending moment

F allowable stress, general

F allowable splice-plate stress multiplied by proportion of panel width actually spliced

F_c allowable stress, compression in plane of plies // stringers

F_c allowable stress, compression // grain in stringers

$F_{c\perp}$ allowable stress, bearing on plywood face

F_s allowable stress, rolling shear

F_t allowable stress, tension in plane of plies // stringers

F_t allowable stress, tension // grain in stringers

F_v allowable stress, horizontal shear, in stringers

G modulus of rigidity in stringers

I moment of inertia, total x-sectional area (finished) of all stringers

I moment of inertia, in direction $\perp$ stringers, of top skin $A_{\perp}$

I_g gross I of total panel x-section about deflection N.A.

I_n gross I of total panel x-section about bending N.A.

I_o gross moment of inertia of x-section portion about own centroidal axis

$I_{//}$, $I_{\perp}$ moment of inertia for plies, corresponding to $A_{//}$ and $A_{\perp}$ areas

ℓ clear span of panel, in direction of stringers

ℓ_e plywood end bearing length required at each end of panel

ℓ' panel width (skins only), perpendicular to ℓ

ℓ'' clear distance between stringers

p design LL or TL (use load related to assumed C factor)

p_a allowable axial load (TL) in beam-column

p_b allowable load (TL) - bending moment

p_d allowable load (TL) - panel deflection

p_{dm} same as p_m but specifically for dynamic loads/loadings

p_m smallest of calculated allowable transverse loads (TL)(in PSSPs for: deflection, bending moment, rolling shear and horizontal shear)

p_p allowable load (TL) - tension splice-plate

p_s allowable load (TL) - rolling shear

p_t allowable load (TL) - top skin deflection

p_v allowable load (TL) - horizontal shear

Q_s statical moment, about neutral axis for deflection, of parallel plies outside critical plane for rolling shear (see A above)

Q_v statical moment, about neutral axis for deflection, of stringers and $A_{//}$ plies x-sectional areas, taken either above or below that axis (used in horizontal shear allowable load calculations)

t glueline width of each stringer (used in $\Sigma F_s t$)

t sum of stringer widths, including side projecting portions

t_h thickness of header (solid across all panel stringers)

y moment arms used in neutral axes calculations

y' half-thickness of parallel plies outside critical plane for rolling shear (see Q_s and A above)

$\bar{y}$ distance from neutral axis to bottom extreme fibre (calculated in both deflection and bending moment calculations for neutral axis)

μ ductility ratio (maximum to elastic deflection, of a selected point, usual at mid-span or mid-height)

REFERENCES

1. "Design of Plywood Stressed-Skin Panels," Supplement 3 to Plywood Design Specification (PDS), American Plywood Association, 1119 A Street, Tacoma, Washington 98401, revised December 1976.

2. Plywood Design Specification (PDS), American Plywood Association, revised December 1976.

3. National Design Specification for Stress-Grade Lumber and Its Fastenings, National Forest Products Association, 1619 Massachusetts Avenue, N.W., Washington, D.C. 20036, 1973 edition, with Table 1 Supplement (allowable unit stresses, published separately), April 1973, revised November 1974.

4. Wood Handbook: Wood as an Engineering Material, Forest Products Laboratory, Forest Service, U.S. Department of Agriculture, Agriculture Handbook No. 72 (Government Printing Office, Washington, D.C.), revised August 1974.

5. Gerhards, C. C., Effect of Duration and Rate of Loading on Strength of Wood and Wood-Based Materials, USDA Forest Service Research Paper FPL 283, 1977, U.S. Forest Products Laboratory, Madison, Wisconsin 53705.

6. Drawsky, R. H., and J. M. Carney (editor), Stressed Skin Panel Tests, Laboratory Report No. 82, Douglas Fir Plywood Association, Tacoma, Washington, April 1960.

7. Fabrication of Plywood Stressed-Skin Panels, Plywood Fabrication Specification SS-8, American Plywood Association, Tacoma, Washington 98401, 1974.

Appendix A2

PLYWOOD STRESSED-SKIN PANELS (TWO-SIDED) AS BEAM-COLUMNS

CONTENTS

TABLES

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Appendix A2

PLYWOOD STRESSED-SKIN PANELS (TWO-SIDED) AS BEAM-COLUMNS

Design

In the preceding Appendix A1, a design procedure, useful stresses, and typical designs were developed for plywood stressed-skin panels (PSSPs) and their estimated ultimate/collapse strength capacity for lateral/transverse (blast) loads. Such panels are of considerable interest to the overall purposes of the project work because abundant supplies of wood for stringers and plywood for skins are available in local lumberyards. Thus their potential is high for use in expedient upgrading of existing basements for shelter against the combined effects of a nuclear weapon detonation. These panels are treated in Appendix A1 in terms of their usefulness as closures, that is to resist transverse blast loads. The purpose of this appendix is to develop procedures for use of such panels as beam-columns, that is to resist axial (blast) loads, without or combined with transverse/lateral (blast) loads.

The basic references of Appendix A1 also contain information pertinent to beam-column design, or simple column design alone [1(sec.3),2].[1] The formula provided for the latter is

$$P_a = 3.619\ (EI_g)\ /\ \ell^2 \qquad (1)$$

or

$$P_a = F_c A \qquad (2)$$

whichever value is less, where[2]

P_a = allowable axial load (lbs), if axial load only exists

(EI_g) = stiffness factor for moment deflection [1(Sec.2.4.3)] (lbs-in.^{2} for full panel)(from Step 4, Appendix A1)

ℓ = clear span of member (simply-supported/pin-ended)(in.)

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[1] Brackets are used herein to indicate sources in the References list at the end of this appendix.

[2] Variables are defined herein at point of first use and in Notation at end of appendix.

F_c = allowable compressive stress (parallel to grain) for plywood skins (psi) [2(p.17)], corrected for buckling [1(Sec.2.5.4)]

A = total x-sectional area of longitudinal grain material in both plywood skins and stringers ($in.^2$)

The interaction formula provided for beam-columns is

$$P/P_a + (M/S) / F_c \leq 1 \qquad (3)$$

where

P = allowable axial load (lbs), under combined loading

M = allowable bending moment (in.-lb), under combined loading

S = I_n / c

in which

S = section modulus of full panel ($in.^3$)

I_n = bending moment of intertia of full panel ($in.^4$)

c = distance from N.A. (bending) to extreme fiber in compression (in.)

Calculations of I_n and c (or $\bar{y}$) are shown in Appendix A1, Figure 3, and Steps 7 and 8.

Assuming that the authors of References [1 and 2] used theory including a solid, rectangular cross-section column, then $I = bd^3/12$. Using this for I_g and solving Equations 1 and 2 for F_c (also recalling that $r^2 = I/A$) leads to

$$F_c = (0.3016\ E) / (\ell/d)^2 = (\pi^2 E) / (2.727(\ell/r)^2)$$

where

I_g = gross moment of inertia of cross section ($in.^4$)

b = least dimension of solid rectangular cross section (in.)

d = greater dimension of solid rectangular cross section (in.)

r = radius of gyration (in.)

E = modulus of elasticity (psi)

The equation's left form is found in Reference [3(p.15 and 65)], under simple solid-column design, indicating that the assumption above is correct. The equation's right form is Euler's equation [4(Eq.3 and 14)] in one of its many forms. Euler's equation is suitable for simply-supported/pin-ended long columns at <u>ultimate</u> (not allowable) load; it is non-conservative [4] when applied to columns with ℓ/r less than about 150,[3] a value much too high for the uses contemplated herein; and the above constant, 2.727, is a factor of safety.

The serious concern with using the foregoing for blast loads is that various approximations have been introduced that can be collectively tolerated because of the allowable/working stress approach for normal uses. Where one is dealing with collapse strength of a column or beam-column, the design approach must take dynamic buckling directly into account and must consider deflection, usually at mid-height, caused by <u>all</u> loads, plus initial eccentricity if it is known or can be estimated. Thus it was concluded that a beam-column design approach should include iteration toward an estimated total deflection from all sources, i.e., initial eccentricity if any, as well as deflection from moments due to transverse and axial loads.[4] The following design approach includes such iteration; it comes from Reference [4(p.5-42, Eq.18)], and is converted to PSSP Notation (Appendix A1 and herein). Henceforth

$$F_c = P/A + (M + P_y)(c/I_n) \qquad (4)$$

where

M_{max} = maximum moment caused by transverse loads only (in.-lb)

y = deflection of column at M (in.)

The referenced source suggests iteration toward a final value for y, using for a first trial value that from M alone[5] in the right-side second term of Equation 4. An approach to performing the suggested iteration follows, using the simply-supported/pin-ended member assumption stated earlier.

From Reference [6], for transverse loads[5] (and modified to Notation herein):

$$M_{mid\text{-}ht} = p_m \, \ell' \, \ell^2 \,/\, 8 \qquad (5)$$

$$\bar{y}_{mid\text{-}ht} = 5 \, p_m \, \ell' \, \ell^4 \,/\, (384(EI_n)) \qquad (6)$$

---

[3] Meaning, for a rectangular column cross-section, ℓ/d less than about 43 (but it's about 24 per Ref.[3],p.65).

[4] An iterative numerical method for analyzing a beam-column is available [5(App.A,p.6-160)].

[5] That is, with P = 0.

where

$\bar{y}_{mid\text{-}ht}$ = deflection of column at M_{max} (transverse loads only)(in.)

Thus, for combined transverse and axial loads:

$$y_{mid\text{-}ht} = 5\ \ell^2\ (M_{mid\text{-}ht} + P_a y)\ /\ (48(EI_n)) \qquad (7)$$

where

p_m = smallest of calculated allowable transverse (only) loads (in PSSPs, calculated loads for: deflection, bending moment, rolling shear and horizontal shear) (psi) (from Appendix A1 design of PSSPs)

$y_{mid\text{-}ht}$ = deflection of column at M (in.)

ℓ' = width of PSSP skins (perpendicular to stringers) (in.)

(EI_n) = stiffness factor for bending moment [1(Sec.2.5.3)] (lb-in.^{2} for full panel) (from Step 8, Appendix A1)

y = trial y at mid-height

For examining locations other than at mid-height of the (prismatic) beam-column, similar equations to Equations 5 to 7 would then be [6]:

$$M_x = p_m \ell' x\ (\ell - x)\ /\ 2 \qquad (8)$$

$$\bar{y}_x = p_m \ell' x\ (\ell^3 - 2\ell x^2 + x^3)\ /\ (24(EI_n)) \qquad (9)$$

$$y_x = (M_x + P_a y)\ (\ell^2 + \ell x - x^2)\ /\ (12(EI_n)) \qquad (10)$$

where

x = location being examined (length along member) (in.)

The overall design approach just described should be applied with due regard to variation in units: some of the parameters are for full panel width, some would usually be applied to design of a one-inch wide strip of panel. All units, therefore, should be checked for values appropriate to one width or the other. All formulas herein are dimensionally consistent; there are no dimensions hidden in constants.

A. Design Procedures

Steps in the design procedure follow.

1. Assume a trial section and clear span/height (in direction of stringers); see Figure 1A, Appendix A1. Use only stress-graded stringers, with face grain of both plywood skins parallel to the stringers. Plan connections to PSSP such that loads are only axial, on pinned ends, with or without uniformly distributed transverse/lateral loads.

2. Same as Step 2, Appendix A1.

3. Calculate A as in Step 3 and Figure 2A, Appendix A1.

4-6. Same as Steps 7 through 9, respectively, of Appendix A1. The c value needed later comes from Step 4 (either $\bar{y}$ in Figure 3A, Appendix A1, or the actual PSSP thickness minus $\bar{y}$, probably the latter but certainly whichever value is for the compression side).

At this point, values for the following variables used in this appendix are known: A (from Step 3), c (4), (EI_n) (5), F_c (6), I_n (5), ℓ (1), and ℓ' (1).

7. Set M = 0 and Equation 4 becomes:

$$F_c = P_a/A + P_a(cy/I_n) \quad \underline{\text{or}} \quad P_a = F_c \,/\, (1/A + cy/I_n) \tag{11}$$

From Equation 2:

$$F_c = P_a \,/\, A \tag{12}$$

Whether $y \geq 0$ (with trial design F_c held constant), the P_a of Equation 12 will be $\geq$ the P_a of Equation 11, thus Equation 11 is used below. This P_a is the maximum allowable axial load, applied when M = 0 (i.e., transverse/lateral load is zero).

8. Set M = 0, then iterate on trial y, using as a minimum value/eccentricity (in.)

$$y = 0.01\,\ell \,/\, d'$$

where

d' = depth of PSSP stringers (in.)

then solving Equation 11 for P_a and Equation 7 with trial y value on right; repeat until y values on right and left sides of Equation 7 are equal or acceptably close (say 1% to 5%).

9. For combined transverse and axial loads, the PSSP must first be investigated, using Steps 1-14, Appendix A1, to find p_m, the peak tran-

sverse load capability with $P_a = 0$; if a dynamic loading p_{dm} value was found, it must be corrected to an equivalent static load p_m:

If the PSSP is one of those pre-designed and shown in Appendix A1, its p_{dm} value may be read from Figures 5-7 there, as the air blast peak overpressure (psi). However, such p_{dm} is based on the Design Stresses-Blast . . . section[6] of the appendix, which includes use of $\mu = 2$ and a step pulse, meaning that the static equivalent p_m is 4/3 the chart value p_{dm} (still with design stresses greatly increased over those for normal, not blast-resistant, use). A value for μ and blast design stresses in beam-columns, in contrast to normal-use design stresses, are discussed in the next section.

Subscripts for mid-height will be dropped from here on, for convenience; the PSSP should be prismatic and with negligible initial eccentricity,[7] therefore all M and y values will be for mid-height (mid-length) for a vertical (horizontal) beam-column.

10. Solve Equations 5 and 6 for M and related y, when P = 0.

From Step 8, values of P_a and related y, when M = 0, are known. Thus the two extreme values of transverse or axial load capacity, with their related mid-height/mid-length deflections, are known at this point. These unique values will be identified as M_{max} (or its related p_m of Step 9) and P_a in the steps below.

11. Assume: a value for P between P_a and zero, and a first trial *value[7] for y proportional* to those found in Steps 8 and 10 (for P_a and for P = 0, respectively).

12. Solve Equation 4 for M:

$$M = (I_n / c)(F_c - P / A) - Py, \quad \text{but} \leq (M_{max} - Py) \tag{13}$$

13. Solve Equation 7 using the trial y on the right side. Compare the left-side y, found from solving Equation 7, with the trial y used. If the two y values are not in acceptable agreement (say, 1% to 5%), use the left-side value as the new trial value[7] of y and *repeat* Steps 12 and 13; otherwise, proceed with the next design step.

14. Find allowable p'_m related to the final M of Step 12:

$$p'_m = (M_{(step\ 12)} / M_{max})\, p_{m(step\ 9)} \tag{14}$$

---

[6] Includes, at the end of that section, a definition of step pulse and the basic relationship $p_{dm} = p_m (1 - 1/(2\mu))$. It follows that $P_{da} = P_a (1 - 1/(2\mu))$, but μ will usually be different in the two uses: (1) lateral loads only; <u>or</u> (2) axial loads only, or axial and lateral loads.

[7] But using not less than the minimum eccentricity value of Step 8.

(This allowable p'_m could also be found using Equation 5 with the final M of Step 12, or Equation 6 with the final left-side y of Step 13.)

15. With allowable P (Step 11) and p'_m (Step 14) known, one pair of pertinent values for the assumed trial section PSSP has been found, besides the two pairs of extreme values (Step 10). Other pairs of values are found by repeating Steps 11-14. To complete the design, a new trial section(s) may have to be assumed, repeating Steps 1-14.

B. Design Stresses (Blast versus Normal Loads) and Ductility

The user of this appendix is referred to a section with the same title, appearing in Appendix A1; the information there is applicable to this appendix except for the last paragraph, which deals with a value for the ductility ratio μ.

For a value of the ductility ratio μ for beam-columns, $\mu = 1$ is recommended for use because of buckling considerations. Increases of normal-use stresses are those already recommended for adoption.[8] If a step pulse (defined in the Appendix A1 section) is appropriate, then the footnote to design Step 9 applies, thus $P_{da} = P_a/2$ ($P_d = P/2$) and $p_{dm} = p_m/2$ ($p'_{dm} = p'_m/2$); p_{dm} or p'_{dm} is the allowable load from peak exterior blast incident overpressure on the PSSP, and p_m or p'_m is the pseudostatic uniform load capacity, respectively.

It is possible that a significant rise time should be applied to the axial blast load but probably not. However, the transverse blast load occurring inside a basement shelter is very likely to have a significant rise time as well as a significant reduction in peak value from the blast peak exterior incident overpressure, due to room filling.[9] If a rough approximation must be suggested it would be that $p_{dm} = p_m$ ($p'_{dm} = p'_m$) where only human-size doorways and typical basement windows constitute the apertures; large openings would indicate use of $p_{dm} = p_m$ times 3/4, even approaching 1/2. This suggested approach attempts to consider both lengthened rise time and reduced peak value of overpressure in terms of that incident on the basement's exterior.

---

[8] See Appendix A1 section with same title as this one: multiply normal-use F_b, F_v, and F_c values by four, but not $F_{c\perp}$ or E values.

[9] See published guidance on design of combined nuclear weapons effects shelter in planned (new) basements, References [5 and 7], especially the latter's Appendix E appearing in Volume 3; the same Appendix E, written by J. R. Rempel, a colleague, was published in an earlier report, Reference [8]. The Appendix E technique was used to produce a short section and two design graphs [7 and 8 (p.8-112 to 8-114)] giving maximum interior pressure and time to reach such pressure, both in terms of V/A (room volume/total aperture area).

Applications

The computer program used (for PSSP design of "beam/slab" type members) in the earlier publication of Appendix A1, was updated to add beam-column and column PSSP design. Table A2-1 shows a listing of the program, as well as a sample problem. The program listed does not automatically increase normal-use design stresses[8] (they must be entered with multiples, if any, already included by the user), and it does not include the dynamic factors for blast loads (section B above, second paragraph).

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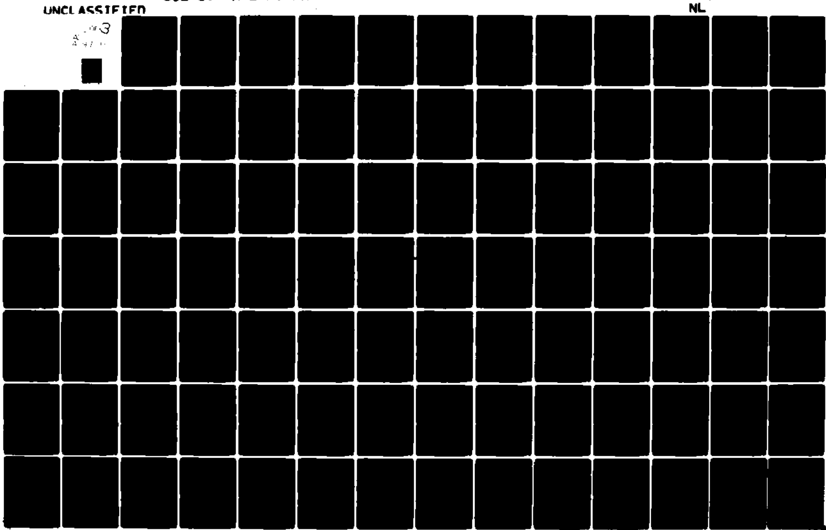

Table A2-1 PSSP DESIGN: COMPUTER PROGRAM (HLMPSP) LISTING AND SAMPLE PROBLEM*

```
LIST
HLMPSP

0050 REM     THIS PROGRAM IS A MODIFICATION AND EXTENSION OF HLMPSP,
0052 REM     TO ADD PROVISION FOR COLUMN AND BEAM-COLUMN DESIGN.
0054 REM
0100 REM    PROGRAM FOR DESIGN OF PLYWOOD STRESSED-SKIN PANELS
0110 REM
0120 REM    PDS = PLYWOOD DESIGN SPEC.,AMER.PLYWOOD ASSN (REV.1/76)
0130 REM    PDS3= SUPPLEMENT 3 (1974) TO PDS
0140 REM    NDS = NATL DES. SPEC. FOR STRESS-GRADE LUMBERS AND ITS
0150 REM      FASTENINGS, TABLE 1 (SUPPLEMENT,REV.11/74)
0160 REM    WDHDBK = WOOD HANDBOOK REV.8/74,U.S.FOREST PRODUCTS LAB
0170 REM    HLMSRI = H.L.MURPHY REPORT DESCRIPTION OF DESIGN PROCEDURE
0180 REM
0181 REM    PROGRAM MODIFIED TO INCLUDE MIN. EXCENTRICITY  11/11/80 - JEB
0185 REM
0190 DIM N[20],A$[10],B$[10]
0191 DIM M[9],P[9],C$[10]
0192 DIM Q[9],Y[9]
0200 GOSUB 540
0210 GOSUB 730
0220 GOSUB 1050
0230 GOSUB 1330
0240 GOSUB 1720
0250 GOSUB 1910
0260 GOSUB 2270
0270 GOSUB 2610
0280 GOSUB 2830
0290 GOSUB 3400
0300 GOSUB 3540
0310 GOSUB 3990
0320 GOSUB 5000
0330 PRINT "ANY CHANGES DESIRED (YES OR NO) "
0340 INPUT B$
0350 IF B$="NO" THEN 430
0360 PRINT "GIVE ONE STEP NUMBER WHERE CHANGE(S) DESIRED "
0370 INPUT N2
0380 IF INT(N2)<1 THEN 360
0390 IF INT(N2)>15 THEN 360
0400 GOSUB N2 OF 540,540,730,1050,1330,1720,1910,2270,2270,2270
0410 GOSUB N2-10 OF 2610,2830,3400,3540,3990
0420 GOTO 330
0430 PRINT
0440 PRINT "WANT TABULATION OF INPUT VALUES (YES OR NO) "
0450 INPUT B$
0460 IF B$="NO" THEN 490
0470 GOSUB 4240
0480 GOTO 320
0490 GOSUB 5000
0500 REM ********************
0510 REM * STEP 2  (HLMSRI) *
0520 REM ********************
0530 GOTO 640
0540 PRINT "STEP 2:"
0550 PRINT "ENTER 'B' DISTANCES FOR TOP & BOT. SKINS, RESP'Y(IN.)"
0560 PRINT "    (HLMSRI, FIG.1B; ALSO PDS3, P.5(TABLE))"
0570 INPUT B1,B2
0580 PRINT
0590 PRINT "ENTER CLEAR DISTANCE BETWEEN STRNGRS (FIG.1A,HLMSRI)(IN.)"
0600 PRINT "    (SHOULD BE UNIFORM; IF NOT, USE LARGEST VALUE)"
0610 INPUT L4
0620 PRINT
0630 RETURN
0640 IF L4 <= 2*B1 THEN 720
0650 IF L4 <= 2*B2 THEN 720
0660 PRINT "WARNING: PROGRAM DESIGN PROCEDURE NOT APPLICABLE"
0670 PRINT "    (HLMSRI, STEP 2; ALSO PDS3,SEC.2.2.1.1,P.5)"
0680 PRINT
0690 REM ********************
0700 REM * STEP 3  (HLMSRI) *
0710 REM ********************
0720 GOTO 930
0730 PRINT "STEP 3:"
0740 PRINT "ENTER E VALUES (PSI) FOR TOP,BOT.SKINS & STRINGERS,RESP'Y"
0750 PRINT "    (PDS,P.17 & NDS)"
0760 INPUT E1,E2,E3
0770 PRINT
0780 PRINT "ENTER A// VALUES FOR TOP,BOT.SKINS (SQ IN/FT WIDTH)"
0790 PRINT "    (PDS,P.16,COL.4)"
0800 INPUT A1,A2
0810 PRINT
0820 PRINT "ENTER TOTAL X-SECT.AREA OF ALL STRINGERS (SQ IN)"
0830 INPUT A3
0840 PRINT
0850 PRINT "ENTER MOMENT ARMS FOR TOP,BOT.SKINS & STRNGRS,RESP'Y(IN.)"
0860 PRINT "    (FROM BOTTOM SURFACE OF PANEL)"
0870 INPUT Y1,Y2,Y3
0880 PRINT
0890 PRINT "ENTER WIDTH (PLYWOOD) OF PANEL (IN.)";
0900 INPUT L2
0910 PRINT
0920 RETURN
0930 LET N[1]=A1/12*L2+A2/12*L2+A3
0940 LET N[2]=A1/12*L2*E1*1.1+A2/12*L2*E2*1.1+A3*E3*1.03
0950 LET N[3]=A1/12*L2*E1*1.1*Y1+A2/12*L2*E2*1.1*Y2+A3*E3*1.03*Y3
0960 LET Y7=N[3]/N[2]
0970 PRINT "STEP 3:"
0980 PRINT "Y-BAR (DEFLECTION N.A.) = ";Y7;"   IN."
0990 PRINT "HLMSRI FIG.2A 'TOTALS': ",N[1],N[2],N[3]
1000 PRINT
1010 REM ********************
1020 REM * STEP 4  (HLMSRI) *
1030 REM ********************
1040 GOTO 1140
1050 PRINT "STEP 4:"
1060 PRINT "ENTER I OF TOP,BOT.SKINS ABOUT OWN AXES(IN.4/FT WIDTH)"
1070 PRINT "    (PDS,P.16,COL.5)"
1080 INPUT I1,I2
1090 PRINT
1100 PRINT "ENTER TOTAL WIDTH OF STRINGERS (IN.) ";
1110 INPUT T5
1120 PRINT
1130 RETURN
1140 LET I3=T5*(A3/T5)^3/12
1150 LET D1=Y1-Y7
1160 LET D2=Y7-Y2
1170 LET D3=ABS(Y7-Y3)
1180 LET N1=I1/12*L2+A1/12*L2*D1^2
1190 LET N2=I2/12*L2+A2/12*L2*D2^2
1200 LET N3=I3+A3*D3^2
1210 LET I7=N1+N2+N3
1220 LET N4=E1*1.1*N1
1230 LET N5=E2*1.1*N2
1240 LET N6=E3*1.03*N3
1250 LET E4=N4+N5+N6
1260 PRINT "STEP 4:"
1270 PRINT "(EI-SUB G) = ";E4;"   LB-IN.2"
1280 PRINT "HLMSRI FIG.2B 'TOTALS :",I7,E4
1290 REM ********************
1300 REM * STEP 5  (HLMSRI) *
1310 REM ********************
1320 GOTO 1380
```

* Sample problem (on last three sheets of this table is the one used in Reference 1.

Table A2-1 (continued)

```
1330 PRINT "STEP 5:"
1340 PRINT "ENTER CLEAR SPAN L OF PANEL (IN.) ";
1350 INPUT L1
1360 PRINT
1370 GOTO 1430
1380 IF L1-INT(L1)<.01 THEN 1410
1390 LET L1=INT(L1)+1
1400 GOTO 1420
1410 LET L1=INT(L1)
1420 GOTO 1530
1430 PRINT "ENTER C FACTOR, IN MAX.ALLOWABLE DEFL.=L/C"
1440 PRINT "    (HLMSRI,STEP6; PDS3,P.9)"
1450 INPUT C1
1460 PRINT
1470 PRINT "IS C FACTOR IN TERMS OF LL OR TL ";
1480 INPUT A$
1490 PRINT
1500 IF A$="TL" THEN 1570
1510 IF A$="LL" THEN 1570
1520 GOTO 1470
1530 LET P1=1/(C1*L1*L2*(5*L1^2/384/E4+.15/A3/(.06*E3*1.03)))
1540 IF A$="TL" THEN 1620
1550 LET N1=P1
1560 GOTO 1610
1570 PRINT "ENTER DESIGN DL (PSI) ";
1580 INPUT P9
1590 PRINT
1600 RETURN
1610 LET P1=P1+P9
1620 PRINT
1630 PRINT "STEP 5:"
1640 PRINT "P-SUB D = ";P1;"   PSI"
1650 IF A$="TL" THEN 1670
1660 PRINT "MADE UP OF LL = ";N1;"   AND DL = ";P9;"   PSI"
1670 PRINT "PANEL DEFL.(MID-SPAN,SS) = ";L1/C1;"   IN."
1680 REM ********************
1690 REM * STEP 6  (HLMSRI) *
1700 REM ********************
1710 GOTO 1780
1720 PRINT "STEP 6:"
1730 PRINT "ENTER I FOR TOP SKIN (STRESS PERPENDICULAR TO STRNGERS)"
1740 PRINT "    (PDS,P.16,COL.9)(IN.4/FT)"
1750 INPUT I8
1760 PRINT
1770 RETURN
1780 IF A$="TL" THEN 1810
1790 LET P7=384*E1*I8/12/C1/L4^3+P9
1800 GOTO 1820
1810 LET P7=384*E1*I8/12/C1/L4^3
1820 PRINT
1830 PRINT "STEP 6:"
1840 PRINT "P-SUB T = ";P7;"   PSI"
1850 PRINT "    TOP-SKIN DEFL.BETW.STRINGERS = ";L4/C1;"   IN."
1860 PRINT
1870 REM ********************
1880 REM * STEP 7  (HLMSRI) *
1890 REM ********************
1900 GOTO 1970
1910 PRINT "STEP 7:"
1920 PRINT "ENTER EFFECTIVE WIDTHS (AS FLANGES) OF TOP, BOT.SKINS"
1930 PRINT "    (HLMSRI,FIG.3A; PDS3,P.10,RT.FIG.)"
1940 INPUT W1,W2
1950 PRINT
1960 RETURN
1970 LET N[5]=A1/12*W1+A2/12*W2+A3
1980 LET N[6]=A1/12*W1*E1*1.1+A2/12*W2*E2*1.1+A3*E3*1.03
1990 LET N[7]=A1/12*W1*E1*1.1*Y1+A2/12*W2*E2*1.1*Y2+A3*E3*1.03*Y3
2000 LET Y8=N[7]/N[6]
2010 PRINT "STEP 7:"
2020 PRINT "Y-BAR (BENDING N.A.) = ";Y8;"   IN."
2030 PRINT "HLMSRI FIG.3A (TOTALS) (PLUS TOTAL OF AREAS):"
2040 PRINT "    ",N[5],N[6],N[7]
2050 PRINT
2060 REM ********************
2070 REM * STEP 8  (HLMSRI) *
2080 REM ********************
2090 LET D4=Y1-Y8
2100 LET D5=Y8-Y2
2110 LET D6=ABS(Y8-Y3)
2120 LET N1=I1/12*W1+A1/12*W1*D4^2
2130 LET N2=I2/12*W2+A2/12*W2*D5^2
2140 LET N3=I3+A3*D6^2
2150 LET I9=N1+N2+N3
2160 LET N4=E1*1.1*N1
2170 LET N5=E2*1.1*N2
2180 LET N6=E3*1.03*N3
2190 LET E5=N4+N5+N6
2200 PRINT "STEP 8:"
2210 PRINT "(EI-SUB N) = ";E5;"   LB-IN.2)"
2220 PRINT "HLMSRI FIG.3B (TOTALS):",I9,E5
2230 REM ********************
2240 REM * STEP 9  (HLMSRI) *
2250 REM ********************
2260 GOTO 2330
2270 PRINT "STEPS 9 & 10:"
2280 PRINT "ENTER TOP SKIN F-SUB C & BOT.SKIN F-SUB T (PSI)"
2290 PRINT "    (PDS,P.17)"
2300 INPUT F1,F2
2310 PRINT
2320 RETURN
2330 IF L4/B1 <= .5 THEN 2450
2340 IF L4/B1 <= 2 THEN 2380
2350 PRINT "WARNING: STRINGER SPACING SHOULD BE <= 2 B (SUB T OR C)"
2360 PRINT "    (B VALUES FROM STEP 2)"
2370 PRINT
2380 IF L4/B1 >= 1 THEN 2410
2390 LET F1=F1*(1-.333*2*(L4/B1-.5))
2400 GOTO 2450
2410 LET F1=F1*.667
2420 REM *********************
2430 REM * STEP 10  (HLMSRI) *
2440 REM *********************
2450 IF L4/B2 <= .5 THEN 2570
2460 IF L4/B2 <= 2 THEN 2500
2470 PRINT "WARNING: STRINGER SPACING SHOULD BE <= 2 B (SUB T OR C)"
2480 PRINT "    (B VALUES FROM STEP 2)"
2490 PRINT
2500 IF L4/B2 >= 1 THEN 2530
2510 LET F2=F2*(1-.333*2*(L4/B2-.5))
2520 GOTO 2570
2530 LET F2=F2*.667
2540 REM *********************
2550 REM * STEP 11  (HLMSRI) *
2560 REM *********************
2570 PRINT
2580 PRINT "STEPS 9 & 10:"
2590 PRINT "F-SUB C AND T,RESP'Y = ";F1;"   AND   ";F2;"   PSI"
2600 GOTO 2660
2610 PRINT "STEP 11:"
2620 PRINT "ENTER OVERALL PANEL THICKNESS (IN.) ";
2630 INPUT T1
2640 PRINT
2650 RETURN
```

Table A2-1 (continued)

```
2660 LET N1=A*(T1-Y8)*L2/L1/2*E5/(E1*1.1)
2670 LET N2=A*Y8*L2/L1/2*E5/(E2*1.1)
2680 LET N[9]=F1*N1
2690 LET N[10]=F2*N2
2700 IF N[9] <= N[10] THEN 2730
2710 LET P2=N[10]
2720 GOTO 2740
2730 LET P2=N[9]
2740 PRINT
2750 PRINT "STEP 11:"
2760 PRINT "P-SUB B = ";P2;"   PSI"
2770 PRINT "(TOP-SKIN = ";N[9];"   & BOT.SKIN = ";N[10];"   PSI"
2780 PRINT
2790 REM *********************
2800 REM * STEP 12  (HLMSRI) *
2810 REM *********************
2820 GOTO 2930
2830 PRINT "STEP 12:"
2840 PRINT "ENTER AREA    PLIES OUTSIDE CRITICAL PLANES,TOP&BOT."
2850 PRINT "   SKINS, RESP'Y (IN.2 / 48 IN.)"
2860 PRINT "   (2ND&6TH COLS, HLMSRI FIG.4 AND PDS3 TABLE P.14)"
2870 INPUT A6,A7
2880 PRINT
2890 PRINT "ENTER Y-PRIME RELATED VALUES(SAME SOURCE,3RD&7TH COLS,IN.)"
2900 INPUT Y9,Y0
2910 PRINT
2920 GOTO 2960
2930 LET Q1=A6*(L2/48)*(T1-Y7-Y9)
2940 LET Q2=A7*(L2/48)*(Y7-Y0)
2950 GOTO 3150
2960 PRINT "ENTER 3 STRINGER WIDTH TOTALS (IN.) IN FOLLOWING ORDER,"
2970 PRINT "   SEPARATED BY COMMAS:"
2980 PRINT "     UNGLUED (PROTRUDING) STRINGER(S) WIDTH (TOTAL IN.),"
2990 PRINT "     GLUELINE (TOTAL IN.) WIDTH OF EXTERIOR STRINGERS"
3000 PRINT "     WHOSE CLEAR DIST.TO PANEL EDGE IS LESS THAN HALF"
3010 PRINT "     CLEAR DIST.BETW.STRINGERS,"
3020 PRINT "     GLUELINE (TOTAL IN.) WIDTH OF ALL OTHER STRINGERS."
3030 INPUT T2,T3,T4
3040 PRINT
3050 IF T5=T2+T3+T4 THEN 3100
3060 PRINT "SUM OF STRINGER WIDTHS NOT EQUAL TO 'TOTAL WIDTH OF"
3070 PRINT "   STRINGERS' ENTERED UNDER STEP 4"
3080 PRINT
3090 GOTO 2960
3100 PRINT "ENTER F-SUB S FOR TOP,BOT. SKINS, RESP'Y (PSI)"
3110 PRINT "   (PDS,P.17)"
3120 INPUT F3,F4
3130 PRINT
3140 RETURN
3150 LET N[11]=F3*(.5*T3+T4)
3160 LET N[12]=F4*(.5*T3+T4)
3170 REM   N(11) AND N(12) ARE SUMMATION F-SUB S TIMES T VALUES
3180 REM     FOR TOP AND BOTTOM SKINS, RESP'Y
3190 LET N[13]=2*N[11]/L1/L2/Q1*E4/(E1*1.1)
3200 LET N[14]=2*N[12]/L1/L2/Q2*E4/(E2*1.1)
3210 IF N[13]>N[14] THEN 3280
3220 LET P3=N[13]
3230 PRINT "STEP 12:"
3240 PRINT "TOP SKIN CONTROLS: P-SUB S = ";P3;"   PSI"
3250 PRINT "   (BOT.SKIN P-SUB S = ";N[14];"   PSI)"
3260 PRINT
3270 GOTO 3360
3280 LET P3=N[14]
3290 PRINT "STEP 12:"
3300 PRINT "BOTTOM SKIN CONTROLS: P-SUB S = ";P3;"   PSI"
3310 PRINT "   (TOP SKIN P-SUB S = ";N[13];"   PSI)"
3320 PRINT
3330 REM *********************
3340 REM * STEP 13  (HLMSRI) *
3350 REM *********************
3360 LET Q4=A3*(1.5*(Y7-2*Y2))/(A3/T5)
3370 LET Q5=A2/12*L2*(Y7-Y2)
3380 LET Q3=Q4+Q5*E2*1.1/(E3*1.03)
3390 GOTO 3460
3400 PRINT "STEP 13:"
3410 PRINT "ENTER ALLOWABLE HORIZ.SHEAR STRESS IN STRNGRS (PSI)"
3420 PRINT "   (NDS)"
3430 INPUT F5
3440 PRINT
3450 RETURN
3460 LET P4=2*F5*T5/(L1*L2*Q3)*(E4/(E3*1.03))
3470 PRINT "STEP 13:"
3480 PRINT "P-SUB V = ";P4;"   PSI"
3490 PRINT "Q-SUB V = ";Q3;"   IN.3"
3500 REM *********************
3510 REM * STEP 14  (HLMSRI) *
3520 REM *********************
3530 GOTO 3640
3540 PRINT "STEP 14:"
3550 PRINT "ENTER ALLOWABLE STRESS IN BEARING ON PLYWOOD BOT.FACE(PSI)"
3560 PRINT "   (PDS,P.17)"
3570 INPUT F6
3580 PRINT
3630 RETURN
3640 REM
3650 REM
3660 LET P5=P2
3670 IF P5 <= P3 THEN 3690
3680 LET P5=P3
3690 IF P5 <= P4 THEN 3730
3700 LET P5=P4
3710 REM
3720 REM
3730 PRINT
3740 PRINT "STEP 14:"
3750 PRINT "P-SUB M = ";P5;"   PSI"
3760 LET L3=L1/(2*(F6/P5-1))
3770 IF L3 >= 1.5 THEN 3800
3780 LET L3=1.5
3790 GOTO 3840
3800 IF 2*L3-INT(2*L3)>.01 THEN 3830
3810 LET L3=INT(2*L3)/2
3820 GOTO 3840
3830 LET L3=INT(2*L3+1/2)/2
3840 PRINT
3850 PRINT "L-SUB E = ";L3;"   IN."
3900 PRINT
3910 REM **********************
3920 REM * STEP 15B  (HLMSRI) *
3930 REM **********************
3940 PRINT "STEP 15:"
3950 PRINT "OBSERVE PRESCRIBED MIN. LENGTHS FOR TENSION SPLICE-"
3960 PRINT "   PLATES(PDS,P.26 TABLE) & OTHER LIMITATIONS(HLMSRI)"
3970 PRINT
3980 GOTO 4090
3990 PRINT "STEP 15:"
4000 PRINT "ENTER ALLOWABLE SPLICE-PLATE MAX.STRESS (PSI)"
4010 PRINT "   (PDS,P.26 TABLE)"
4020 INPUT F7
4030 PRINT
4040 PRINT "ENTER TOTAL LENGTH OF SPLICE-PLATE ACROSS PANEL (IN.)"
4050 PRINT "   (HLMSRI,STEP 15B; PDS3,P.4 FIG.)"
```

Table A2-1 (continued)

```
4060  INPUT L5
4070  PRINT
4080  RETURN
4090 IF L5<L2 THEN 4130
4100 PRINT "SPLICE-PLATE LENGTH MUST BE SHORTER THAN PANEL WIDTH"
4110  PRINT
4120  GOSUB 4040
4130  LET P6=8*(F7*L5/L2)/Y7/L2/L1^2*E4/(E2*1.1)
4140  PRINT "P-SUB P = ";P6;"   PSI"
4150  PRINT "IF P-SUB P < P-SUB M (";P5;") OR DESIGN TL, WHICHEVER"
4160  PRINT "   CRITERION IS USED, SPLICE-PLATE SHOULD BE REDESIGNED"
4170  PRINT "   OR RELOCATED (P-SUB P CALC'D ON MID-SPAN LOCATION)"
4180  GOSUB 5000
4182  PRINT "RUN A BEAM-COLUMN DESIGN (YES OR NO) "
4183  INPUT B$
4184  IF B$="NO" THEN 4190
4185  PRINT "ARE TRANSVERSE (LATERAL) LOADS ZERO (YES OR NO) "
4186  INPUT C$
4187  GOSUB 5000
4188  GOSUB 6000
4189  GOSUB 5000
4190  PRINT "RUN ANOTHER PROBLEM (YES OR NO) "
4200  INPUT B$
4210  IF B$="NO" THEN 9999
4220  GOSUB 5000
4230  GOTO 200
4240  GOSUB 5000
4250  PRINT "STEP"
4260  PRINT "  2","TOP B","BOTTOM B","CL.DIST.BETW.STRNGR'S"
4270  PRINT "   ",B1,B2,L4
4280  PRINT
4290  PRINT "  3","E TOP SKIN","E BOT.SKIN","E STRNGRS","A// TOP"
4300  PRINT "  ",E1,E2,E3,A1
4310  PRINT "  ","A// BOT.SKIN","AREA STRNGRS","TP.SK.MOM.ARM","BOT.DITTO"
4320  PRINT "  ",A2,A3,Y1,Y2
4330  PRINT "  ","MOM.ARM STR'S","PANEL WIDTH (PLYWOOD)"
4340  PRINT "  ",Y3,L2
4350  PRINT
4360  PRINT "  4","I TOP SK.","I BOT.SK.","TOTAL WIDTH STRNGR'S"
4370  PRINT "   ",I1,I2,T5
4380  PRINT
4390  PRINT "  5","PANEL CL.SP.","C FACTOR","C IN LL OR TL?"
4400  PRINT "   ",L1,C1,A$
4410  PRINT
4420  IF A$="TL" THEN 4470
4430  PRINT "  6","DESIGN DL","I TOP SK.(STRESS PERPEND. TO STR'S)"
4440  PRINT "  ",P9,I8
4450  PRINT
4460  GOTO 4500
4470  PRINT "  6","I TOP SK.(STRESS PERPEND. TO STRINGERS)"
4480  PRINT "  ",I8
4490  PRINT
4500  PRINT "  7","TOP SKIN","& BOT. SKIN","EFFECTIVE WIDTHS"
4510  PRINT "  ",W1,W2
4520  PRINT
4530  PRINT " 9&10","TOP SK.F-SUB C","BOT.SK.F-SUB C"
4540  PRINT "   ",F1,F2
4550  PRINT
4560  PRINT " 11","OVERALL PANEL THICKNESS"
4570  PRINT "  ",T1
4580  PRINT
4590  PRINT " 12","TOP SK.A//","& BOT.SK.A//","BOTH OUTSIDE CRITICAL PL."
4600  PRINT "  ",A6,A7
4610  PRINT "  ","RELATED Y-PRIME VALUES"
4620  PRINT "  ",Y9,Y0
4630  PRINT "  ","UNGLUED","& EXT.GLUED","& INT. GLUED",
4640  PRINT "STRNR WIDTHS"
4650  PRINT "  ",T2,T3,T4
4660  PRINT "  ","TOP SK.F-SUB S","BOT.SK.F-SUB S"
4670  PRINT "  ",F3,F4
4680  PRINT
4690  PRINT " 13","ALLOW.HORIZ.SHEAR STRESS, STRINGERS"
4700  PRINT "  ",F5
4710  PRINT
4720  PRINT " 14","PLYW.BOT.FACE ALLOW.BEAR.STRESS"
4730  PRINT "  ","  ",F6
4740  PRINT
4750  PRINT " 15","ALLOW.SPL-PL.MAX.STRESS","TOTAL X-PANEL SPL-PL.LENGTH"
4760  PRINT "  ",F7,"  ",L5
4770  RETURN
5000   FOR N9=1 TO 5
5010   PRINT
5020   NEXT N9
5030  RETURN
6000  REM      SUBROUTINE TO ADD BEAM-COLUMN (BC) DESIGN CAPABILITY
6010  REM      SEE HLMSRI REPORT OF 10/77 PP. A2-4 TO -6 FOR BC DESIGN STEPS.
6020  REM      BC DESIGN STEPS 1-6 ARE COVERED BY HLMPSP DESIGN STEPS 1-9.
6030  REM      BC STEP 6 CLOSES, REQUIRING VALUES FOR 7 VARIABLES; FIRST 2
6040  REM      ARE PROVIDED BY STATEMENTS FOLLOWING THESE REM'S, AND OTHER 5
6050  REM      ARE: E5 (EI-SUB N), F1 (F-SUB C), I9 (I-SUB N), L1 (L), AND
6060  REM      L2 (L-PRIME), ALL FROM PRECEDING HLMPSP STEPS.
6070  REM
6080  REM      PANEL DEFLECTION AND DEFLECTION BETWEEN STRINGERS HAVE BEEN
6090  REM      EXCLUDED AS TRANSVERSE LOAD CAPACITY CRITERIA, BY LINES 3640-3720
6100  REM      OF THIS MODIFICATION; DELETE THEM IF SUCH CRITERIA EXCLUSION IS
6110  REM      UNDESIRED.
6120  REM
6130  LET A4=A1+A2+A3
6140  LET C2=T1-Y8
6150  REM *************************
6160  REM * STEPS 7-8 BC (HLMSRI) *
6170  REM *************************
6180  LET Y[1]=N[16]=.01*L1/(A3/T5)
6190   FOR J=1 TO 50
6200   LET P[1]=F1/(1/A4+N[16]*C2/I9)
6210   LET N[17]=5*L1^2*P[1]*N[16]/48/E5
6212    IF N[17]>Y[1] THEN 6220
6214   LET N[17]=Y[1]
6220    IF ABS(N[17]-N[16])<.01 THEN 6270
6230   LET N[16]=N[17]
6240    IF J<50 THEN 6260
6250   PRINT "J=50 IN STEP 8 THUS TRIAL/ERROR SOLUTION EQ.A2-7 UNSATISFACTORY"
6260   NEXT J
6270  LET Y[1]=N[17]
6280  IF C$="YES" THEN 6730
6290  REM **********************
6300  REM * STEP 9 BC (HLMSRI) *
6310  REM **********************
6320  REM      P-SUB M IS P5 OF HLMPSP STEP 14 ABOVE
6330  REM      FOR MU: USE U1 FOR AXIAL AND U2 FOR TRANSVERSE (LATERAL) LOADS
6340  LET U1=1
6350  LET U2=1
6360  REM      WHERE P-SUB DM = P-SUB M * (1 - 1/(2*U2))
6370  REM
6380  REM      BUT, DEPENDING ON INTERIOR RISE-TIME, PERHAPS REDUCED PEAK BLAST
6390  REM      PRESSURE, ETC., U2 = 2 ... 1000 AS SUCH THINGS REDUCE THE BLAST
6400  REM      LOADING TOWARDS A STATIC LOAD, I.E., WITH UNITY DYNAMIC
6410  REM      MAGNIFICATION FACTOR.
6420  REM
6430  REM      SEE REPORT TEXT (REVISED APP. A2 IN NEW REPORT).
6440  REM ***********************
6450  REM * STEP 10 BC (HLMSRI) *
```

Table A2-1 (continued)

```
6460 REM ************************
6480 LET M[9]=P5*L2*L1^2/8
6490 LET Y[9]=5*P5*L2*L1^4/384/E5
6500 LET Q[9]=P5
6510 LET M[1]=Q[1]=P[9]=0
6520 REM ****************************
6530 REM * STEPS 11-15 BC (HLMSRI) *
6540 REM ****************************
6550  FOR I=8 TO 2 STEP -1
6560  LET P[I]=P[1]*(9-I)/8
6570  LET N[16]=Y[9]*(I-1)/8
6580   FOR J=1 TO 50
6590   LET M[I]=I9/C2*(F1-P[I]/A4)-P[I]*N[16]
6600   IF M[I] <= M[9]-P[I]*N[16] THEN 6620
6610   LET M[I]=M[9]-P[I]*N[16]
6620   LET N[17]=5*L1^2*(M[I]+P[I]*N[16])/48/E5
6630   IF ABS(N[17]-N[16])<.01 THEN 6680
6640   LET N[16]=N[17]
6650   IF J<50 THEN 6670
6660   PRINT "J=50 IN STEP 13 THUS TRIAL/ERROR SOLUTION EQ.A2-7 UNSATISFACTORY"
6670   NEXT J
6680  LET Y[I]=N[17]
6690  LET Q[I]=M[I]/M[9]*Q[9]
6700  NEXT I
6710 REM      OUTPUT PORTION
6720 PRINT "STEPS 7-8 BC:"
6730 PRINT "MAX.AXIAL LOAD (W/ ZERO LATERAL LOAD),P-SUB A= ";
6740 PRINT INT(P[1]+1/2);" LBS."
6750 IF C$="YES" THEN 4189
6760 PRINT "RELATED MID-LENGTH DEFL. Y=";INT(Y[1]*10000+1/2)/10000;"IN."
6770 PRINT
6780 PRINT "STEPS 9-10 BC:"
6790 PRINT "MAX.MID-LENGTH MOMENT FROM LATERAL LOADS (WITH ZERO AXIAL LOAD)"
6800 PRINT "   ";INT(M[9]/100+1/2)/10;
6810 PRINT "  IN.-KIPS (KIP=KILO-POUND=1,000 LBS)"
6820 PRINT "RELATED DEFLECTION= ";INT(Y[9]*100+1/2)/100;" IN.,MID-LENGTH"
6830 PRINT "RELATED LATERAL LOADING, P-SUB M= ";INT(Q[9]*1000+1/2)/1000;
6840 PRINT " PSI"
6850 PRINT
6860 PRINT "STEPS 11-15 BC:"
6870 PRINT "AXIAL LOAD  LATERAL LOAD/MOMENT  P-DELTA MOM.  DEFLECTION"
6880 PRINT "  (LBS.)       (PSI)   (IN.-KIPS)    (IN.-KIPS)      (IN.)"
6890 PRINT
6900  FOR I=1 TO 9
6910  PRINT USING "#,7D";INT(P[I]+1/2)
6920  PRINT USING "#,2X7D.D";INT(Q[I]*10+1/2)/10
6930  PRINT USING "#,2X7D.D";INT(M[I]/100+1/2)/10
6940  PRINT USING "#,2X9D.D";INT(P[I]*Y[I]/100+1/2)/10
6950  PRINT USING "2X7D.DD";INT(Y[I]*100+1/2)/100
6960  NEXT I
6970 GOSUB 5000
6980 LET T=.0027*L1^2*(144*P9*L2/E5)^.5
6990 PRINT "SIMPLY SUPPORTED, EFFECTIVE NATURAL PERIOD OF (LATERAL) ";
7000 PRINT "VIBRATION,"
7010 PRINT "T= ";INT(T*100000.+1/2)/100000.;" SEC."
7020 RETURN
9999 END
```

SAMPLE PROBLEM (as used in Ref. 1)

```
STEP 2:
ENTER 'B' DISTANCES FOR TOP & BOT. SKINS, RESP'Y(IN.)
   (HLMSRI, FIG.1B; ALSO PDS3, P.5(TABLE))
?32,12

ENTER CLEAR DISTANCE BETWEEN STRNGRS (FIG.1A,HLMSRI)(IN.)
   (SHOULD BE UNIFORM; IF NOT, USE LARGEST VALUE)
?13.9

STEP 3:
ENTER E VALUES (PSI) FOR TOP,BOT.SKINS & STRINGERS,RESP'Y
   (PDS,P.17 & NDS)
?1800000,1800000,1800000

ENTER A// VALUES FOR TOP,BOT.SKINS (SQ IN/FT WIDTH)
   (PDS,P.16,COL.4)
?2.728,1.914

ENTER TOTAL X-SECT.AREA OF ALL STRINGERS (SQ IN)
?32.2

ENTER MOMENT ARMS FOR TOP,BOT.SKINS & STRNGRS,RESP'Y(IN.)
   (FROM BOTTOM SURFACE OF PANEL)
?6,.156,3

ENTER WIDTH (PLYWOOD) OF PANEL (IN.)?48

STEP 4:
ENTER I OF TOP,BOT.SKINS ABOUT OWN AXES(IN.4/FT WIDTH)
   (PDS,P.16,COL.5)
?.141,.025

ENTER TOTAL WIDTH OF STRINGERS (IN.) ?6

STEP 5:
ENTER CLEAR SPAN L OF PANEL (IN.) ?168

ENTER C FACTOR, IN MAX.ALLOWABLE DEFL.=L/C
   (HLMSRI,STEP6; PDS3,P.9)
?360

IS C FACTOR IN TERMS OF LL OR TL ?LL

ENTER DESIGN DL (PSI) ?.0694444

STEP 6:
ENTER I FOR TOP SKIN (STRESS PERPENDICULAR TO STRNGERS)
   (PDS,P.16,COL.9)(IN.4/FT)
?.023

STEP 7:
ENTER EFFECTIVE WIDTHS (AS FLANGES) OF TOP, BOT.SKINS
   (HLMSRI,FIG.3A; PDS3,P.10,RT.FIG.)
?48,42.3

STEPS 9 & 10:
ENTER TOP SKIN F-SUB C & BOT.SKIN F-SUB T (PSI)
   (PDS,P.17)
?1540,1650

STEP 11:
ENTER OVERALL PANEL THICKNESS (IN.) ?6.312
```

Table A2-1 (continued)

```
STEP 12:
ENTER AREA // PLIES OUTSIDE CRITICAL PLANES,TOP&BOT.
   SKINS, RESP'Y (IN.2 / 48 IN.)
    (2ND&6TH COLS, HLMSRI FIG.4 AND PDS3 TABLE P.14)
?6.06,3.83

ENTER Y-PRIME RELATED VALUES(SAME SOURCE,3RD&7TH COLS,IN.)
?.148,.0479

ENTER 3 STRINGER WIDTH TOTALS (IN.) IN FOLLOWING ORDER,
   SEPARATED BY COMMAS:
     UNGLUED (PROTRUDING) STRINGER(S) WIDTH (TOTAL IN.),
     GLUELINE (TOTAL IN.) WIDTH OF EXTERIOR STRINGERS
     WHOSE CLEAR DIST.TO PANEL EDGE IS LESS THAN HALF
     CLEAR DIST.BETW.STRINGERS,
     GLUELINE (TOTAL IN.) WIDTH OF ALL OTHER STRINGERS.
?.75,2.25,3

ENTER F-SUB S FOR TOP,BOT. SKINS, RESP'Y (PSI)
   (PDS,P.17)
?48,48

STEP 13:
ENTER ALLOWABLE HORIZ.SHEAR STRESS IN STRNGRS (PSI)
   (NDS)
?95

STEP 14:
ENTER ALLOWABLE STRESS IN BEARING ON PLYWOOD BOT.FACE(PSI)
   (PDS,P.17)
?340

STEP 15:
ENTER ALLOWABLE SPLICE-PLATE MAX.STRESS (PSI)
   (PDS,P.26 TABLE)
?1200

ENTER TOTAL LENGTH OF SPLICE-PLATE ACROSS PANEL (IN.)
   (HLMSPI,STEP 15B; PDS3,P.4 FIG.)
?40.2


ANY CHANGES DESIRED (YES OR NO)
?NO

WANT TABULATION OF INPUT VALUES (YES OR NO)
?YES
```

```
STEP
 2       TOP B           BOTTOM B         CL.DIST.BETW.STRNGR'S
          32              12               13.9

 3       E TOP SKIN      E BOT.SKIN       E STRNGRS         A// TOP
          1.80000E+06     1.80000E+06      1.80000E+06       2.728
         A// BOT.SKIN    AREA STRNGRS     TP.SK.MOM.ARM     BOT.DITTO
          1.914           32.2             6                 .156
         MOM.ARM STR'S   PANEL WIDTH (PLYWOOD)
          3               48

 4       I TOP SK.       I BOT.SK.        TOTAL WIDTH STRNGR'S
          .141            .025             6

 5       PANEL CL.SP.    C FACTOR         C IN LL OR TL?
          168             360              LL

 6       DESIGN DL       I TOP SK.(STRESS PERPEND. TO STR'S)
          6.94444E-02     .023

 7       TOP SKIN        & BOT. SKIN      EFFECTIVE WIDTHS
          48              42.3

 9&10    TOP SK.F-SUB C  BOT.SK.F-SUB T
          1540            1650

 11      OVERALL PANEL THICKNESS
          6.312

 12      TOP SK.A//      & BOT.SK.A//     BOTH OUTSIDE CRITICAL PL.
          6.06            3.83
         RELATED Y-PRIME VALUES
          .148            .0479
         UNGLUED         & EXT.GLUED      & INT. GLUED    STRNR WIDTHS
          .75             2.25             3
         TOP SK.F-SUB S  BOT.SK.F-SUB S
          48              48

 13      ALLOW.HORIZ.SHEAR STRESS, STRINGERS
          95

 14      PLYW.BOT.FACE ALLOW.BEAR.STRESS
                          340

 15      ALLOW.SPL-PL.MAX.STRESS          TOTAL X-PANEL SPL-PL.LENGTH
          1200                             40.2


ANY CHANGES DESIRED (YES OR NO)
?NO

WANT TABULATION OF INPUT VALUES (YES OR NO)
?NO
```

Table A2-1 (concluded)

```
STEP 3:
Y-BAR (DEFLECTION N.A.) =  3.22501        IN.
HLMSRI FIG.2A 'TOTALS':          50.768            9.64634E+07     3.11096E+08

STEP 4:
(EI-SUB 6) =  4.56775E+08        LB-IN.2
HLMSRI FIG.2B 'TOTALS':          235.716           4.56775E+08

STEP 5:
P-SUB D =  .476408         PSI
MADE UP OF LL =  .406964          AND DL =  6.94444E-02        PSI
PANEL DEFL.(MID-SPAN,SS) =  .466667          IN.

STEP 6:
P-SUB T =  1.43971          PSI
   TOP-SKIN DEFL.BETW.STRINGERS =  3.86111E-02       IN.

STEP 7:
Y-BAR (BENDING N.A.) =  3.28337        IN.
HLMSRI FIG.3A 'TOTALS' (PLUS TOTAL OF AREAS):
                     49.8588         9.46633E+07    3.10815E+08

STEP 8:
(EI-SUB N) =  4.39474E+08        LB-IN.2)
HLMSRI FIG.3B 'TOTALS':          227.039           4.39474E+08

STEPS 9 & 10:
F-SUB C AND T,RESP'Y =  1540        AND     1100.55        PSI

STEP 11:
P-SUB B =  .439327          PSI
TOP-SKIN =  .666459         & BOT.SKIN =  .439327         PSI

STEP 12:
TOP SKIN CONTROLS: P-SUB S =  .63608          PSI
   (BOT.SKIN P-SUB S =  .931003         PSI)

STEP 13:
P-SUB V =  .678801          PSI
Q-SUB V =  51.3103          IN.3

STEP 14:
P-SUB M =  .439327          PSI

L-SUB E =  1.5              IN.

STEP 15:
OBSERVE PRESCRIBED MIN. LENGTHS FOR TENSION SPLICE-
   PLATES(PDS,P.26 TABLE) & OTHER LIMITATIONS(HLMSRI)

P-SUB P =  .424524          PSI
IF P-SUB P < P-SUB M ( .439327     ) OR DESIGN TL, WHICHEVER
   CRITERION IS USED, SPLICE-PLATE SHOULD BE REDESIGNED
   OR RELOCATED (P-SUB P CALC'D ON MID-SPAN LOCATION)


RUN A BEAM-COLUMN DESIGN (YES OR NO)
?YES
ARE TRANSVERSE (LATERAL) LOADS ZERO (YES OR NO)
?NO

STEPS 7-8 BC:
MAX.AXIAL LOAD (W/ ZERO LATERAL LOAD),P-SUB A=  49172.        LBS.
RELATED MID-LENGTH DEFL. Y= .313        IN.

STEPS 9-10 BC:
MAX.MID-LENGTH MOMENT FROM LATERAL LOADS (WITH ZERO AXIAL LOAD)
   74.4          IN.-KIPS (KIP=KILO-POUND=1,000 LBS)
RELATED DEFLECTION=  .5                IN.,MID-LENGTH
RELATED LATERAL LOADING, P-SUB M=  .439          PSI

STEPS 11-15 BC:
AXIAL LOAD  LATERAL  LOAD/MOMENT   P-DELTA MOM.   DEFLECTION
  (LBS.)     (PSI)   (IN.-KIPS)    (IN.-KIPS)       (IN.)

  49172       0.0        0.0          15.4          0.31
  43025       0.1       19.9           8.0          0.19
  36879       0.2       30.4          10.0          0.27
  30732       0.2       42.0          10.9          0.35
  24586       0.3       54.7          10.8          0.44
  18439       0.4       65.2           9.2          0.50
  12293       0.4       68.3           6.1          0.50
   6146       0.4       71.3           3.1          0.50
      0       0.4       74.4           0.0          0.50


SIMPLY SUPPORTED, EFFECTIVE NATURAL PERIOD OF (LATERAL) VIBRATION,
T=  .07964        SEC.


RUN ANOTHER PROBLEM (YES OR NO)
?NO

DONE
BYE
```

The PSSP designs in Table A2-2* show the results of using the computer program (Table A2-1) for columns and beam-columns. These results include use of the recommended increases in normal-use design stresses[8] and use of the recommended dynamic factors for blast loads (section B above, second paragraph).

An illustrative example is shown on the page following Table A2-2.

* Prepared by J. E. Beck of James E. Beck and Associates (subcontractor).

Table A2-2A PSSP DESIGNS FOR LOWER STRENGTH STRINGERS (F_v = 280 psi)
(Columns and Beam-Columns)

Panel Width: 48 in.

Top & Bottom Skins		Stringers		Req'd* Bearing Length Ea. End	Panel Height = 7 ft					Panel Heigth = 8 ft					Panel Height = 9 ft				
					P_{da}# (kips)	P_{dm}(psi) for P_d/P_{da} =				P_{da}# (kips)	P_{dm}(psi) for P_d/P_{da} =				P_{da}# (kips)	P_{dm}(psi) for P_d/P_{da} =			
Nom. Thick.	Face Ply	Nom. Size	No.			0.8	0.6	0.4	0.2		0.8	0.6	0.4	0.2		0.8	0.6	0.4	0.2
1/2	#1	2x4	4	4.5	130.7	0.6	0.9	1.0	1.2	128.5	0.4	0.7	0.9	1.0	126.5	0.2	0.5	0.7	0.8
			5	4.0	169.5	0.6	1.0	1.2	1.4	166.4	0.4	0.7	0.9	1.1	163.4	0.2	0.5	0.7	1.0
			7	3.5	219.5	0.6	1.1	1.4	1.7	214.8	0.3	0.7	1.1	1.4	191.1	0.2	0.5	0.9	1.2
			9	3.5	268.0	0.5	1.1	1.6	2.0	261.5	0.2	0.7	1.2	1.7	203.1	0.3	0.7	1.1	1.5
1/2	#3	2x4	4	4.5	84.0	0.4	0.7	1.0	1.1	82.6	0.3	0.5	0.8	0.9	81.3	0.2	0.4	0.6	0.8
			5	4.0	109.0	0.4	0.8	1.1	1.3	107.0	0.3	0.5	0.9	1.1	105.0	0.1	0.3	0.6	0.9
			7	3.5	141.1	0.4	0.8	1.3	1.6	138.1	0.2	0.5	0.9	1.4	135.1	0.1	0.3	0.6	1.0
			9	3.0	172.3	0.4	0.8	1.3	1.9	168.1	0.2	0.5	0.9	1.5	148.5	0.1	0.4	0.7	1.2
3/4	#1	2x4	4	5.0	152.3	0.8	1.1	1.2	1.3	150.0	0.5	0.9	1.0	1.1	147.9	0.4	0.7	0.8	1.0
			5	4.5	176.5	0.8	1.2	1.4	1.6	175.5	0.5	1.0	1.1	1.3	172.7	0.3	0.7	0.9	1.1
			7	4.0	229.3	0.8	1.5	1.7	2.0	224.8	0.5	1.0	1.4	1.7	220.5	0.3	0.6	1.1	1.4
			9	4.0	276.5	0.8	1.5	2.0	2.3	272.2	0.4	0.9	1.6	2.0	264.2	0.2	0.6	1.2	1.7
3/4	#3	2x4	4	5.0	97.9	0.5	0.9	1.2	1.3	96.4	0.4	0.7	1.0	1.1	95.1	0.2	0.5	0.8	0.9
			5	4.5	114.7	0.5	1.0	1.3	1.5	112.9	0.4	0.7	1.1	1.3	111.0	0.2	0.5	0.8	1.1
			7	4.0	147.4	0.5	1.0	1.5	1.9	144.5	0.3	0.7	1.1	1.6	141.7	0.2	0.5	0.8	1.2
			9	3.5	179.0	0.5	1.0	1.6	2.2	175.0	0.3	0.7	1.1	1.7	171.2	0.2	0.4	0.8	1.3
1-1/8	#1	2x4	4	5.5	170.1	1.2	1.4	1.5	1.6	168.0	0.8	1.1	1.2	1.4	166.0	0.6	0.9	1.1	1.2
			5	5.0	197.2	1.2	1.6	1.7	1.9	194.5	0.8	1.3	1.5	1.6	191.9	0.6	1.1	1.2	1.4
			7	5.0	249.8	1.2	2.0	2.2	2.4	245.7	0.8	1.5	1.8	2.1	241.8	0.6	1.1	1.5	1.8
			9	4.5	300.8	1.2	2.2	2.6	2.9	295.1	0.8	1.5	2.1	2.5	289.6	0.5	1.1	1.8	2.1

* For full lateral load and zero axial load; use 20% axial load (i.e., $P_d = 0.2P_{da}$). Reduce required bearing length (each end) with larger axial loads, varying in accordance with change in p_{dm} values; use minimum of 2 in. each end, however. For example, using data for 1/2 in. #3, 2x4s, 4-stringer, 7-ft long PSSP, and axial load = $0.6P_{da}$: required bearing length each end is 4.5(0.7/1.1) = 2.9 in. each end. The bearing length values were obtained from Table A1-1A.

\# P_{da} is the maximum axial load, that is, when lateral load, p_{dm} = 0.

Table A2-2A (Concluded)

Panel Width: 48 in.

Top & Bottom Skins		Stringers		Req'd* Bearing Length Ea. End	Panel Height = 7 ft					Panel Height = 8 ft					Panel Height = 9 ft				
					P_{da}# (kips)	P_{dm}(psi) for P_d/P_{da} =				P_{da}# (kips)	P_{dm}(psi) for P_d/P_{da} =				P_{da}# (kips)	P_{dm}(psi) for P_d/P_{da} =			
Nom. Thick.	Face Ply	Nom. Size	No.			0.8	0.6	0.4	0.2		0.8	0.6	0.4	0.2		0.8	0.6	0.4	0.2
1/2	#1	2x6	4	7.5	201.2	1.1	1.7	1.9	2.0	199.4	0.8	1.4	1.6	1.7	197.6	0.6	1.1	1.3	1.5
			5	6.5	266.7	1.2	2.0	2.2	2.4	264.0	0.8	1.6	1.8	2.0	261.3	0.6	1.2	1.5	1.8
			7	6.0	356.2	1.3	2.5	2.8	3.1	352.0	0.9	1.8	2.3	2.7	347.8	0.6	1.2	1.9	2.3
			9	5.5	444.6	1.4	2.8	3.4	3.8	438.7	0.9	1.9	2.8	3.2	433.0	0.6	1.3	2.3	2.8
1/2	#3	2x6	4	7.0	129.4	0.7	1.3	1.8	1.9	128.2	0.5	1.0	1.5	1.7	127.0	0.4	0.7	1.2	1.4
			5	6.5	171.4	0.8	1.5	2.1	2.3	169.7	0.6	1.1	1.7	2.0	168.0	0.4	0.6	1.3	1.7
			7	5.5	229.0	0.9	1.7	2.6	3.0	226.3	0.6	1.2	1.9	2.6	223.6	0.4	0.9	1.4	2.1
			9	5.5	285.8	1.0	1.9	2.9	3.7	282.0	0.7	1.3	2.1	3.1	278.4	0.4	0.9	1.6	2.4
3/4	#1	2x6	4	8.0	229.7	1.4	1.9	2.0	2.1	227.8	1.0	1.6	1.7	1.8	225.9	0.7	1.3	1.4	1.6
			5	7.0	275.3	1.4	2.2	2.4	2.6	272.8	1.0	1.8	2.0	2.2	270.3	0.7	1.4	1.7	1.9
			7	6.5	365.3	1.6	2.8	3.1	3.4	361.3	1.1	2.1	2.6	2.9	357.3	0.7	1.5	2.2	2.5
			9	6.0	454.1	1.7	3.1	3.8	4.1	448.4	1.1	2.2	3.1	3.5	442.9	0.7	1.6	2.6	3.0
3/4	#3	2x6	4	7.5	147.7	0.9	1.7	2.0	2.1	146.5	0.7	1.3	1.7	1.8	145.3	0.5	0.9	1.4	1.6
			5	7.0	177.0	1.0	1.8	2.3	2.5	175.4	0.7	1.3	2.0	2.1	173.7	0.5	1.0	1.5	1.9
			7	6.0	234.9	1.1	1.9	3.0	3.3	232.3	0.7	1.4	2.2	2.8	229.7	0.5	1.0	1.7	2.4
			9	5.5	291.9	1.1	2.1	3.2	4.0	288.3	0.8	1.5	2.4	3.4	284.7	0.5	1.1	1.8	2.6
1-1/8	#1	2x6	4	8.5	247.3	1.9	2.2	2.3	2.4	245.6	1.4	1.8	1.9	2.1	243.9	1.0	1.6	1.7	1.8
			5	8.0	293.6	1.9	2.6	2.7	2.9	291.3	1.4	2.2	2.3	2.5	289.0	1.1	1.8	2.0	2.2
			7	7.0	384.8	2.1	3.4	3.6	3.8	381.2	1.5	2.8	3.0	3.3	377.6	1.1	2.1	2.6	2.9
			9	6.5	474.6	2.2	4.0	4.4	4.7	469.5	1.5	2.9	3.7	4.0	464.4	1.1	2.1	3.1	3.5
1-1/8	#1	2x8	4	11.0	313.7	2.6	3.0	3.1	3.2	312.2	1.9	2.5	2.6	2.7	310.7	1.5	2.2	2.3	2.4
			5	10.5	376.7	2.7	3.6	3.7	3.9	374.6	2.0	3.0	3.2	3.4	372.6	1.5	2.6	2.8	2.9
			7	9.5	501.5	3.0	4.8	5.0	5.2	498.3	2.2	4.0	4.3	4.5	495.1	1.6	3.2	3.7	4.0
			9	9.0	625.3	3.2	5.9	6.2	6.6	620.7	2.4	4.5	5.3	5.7	616.2	1.7	3.4	4.5	4.9

Table A2-2B PSSP DESIGNS FOR HIGHER STRENGTH STRINGERS (F_v = 380 psi)
(Columns and Beam-Columns)

Panel Width: 48 in.

Top & Bottom Skins		Stringers		Req'd* Bearing Length Ea. End	Panel Height = 7 ft					Panel Height = 8 ft					Panel Height = 9 ft				
					P_{da}# (kips)	p_{dm}(psi) for P_d/P_{da} =				P_{da}# (kips)	p_{dm}(psi) for P_d/P_{da} =				P_{da}# (kips)	p_{dm}(psi) for P_d/P_{da} =			
Nom. Thick.	Face Ply	Nom. Size	No.			0.8	0.6	0.4	0.2		0.8	0.6	0.4	0.2		0.8	0.6	0.4	0.2
1/2	#1	2x4	4	3.5	130.7	0.6	1.1	1.4	1.6	128.5	0.4	0.7	1.2	1.3	126.5	0.2	0.5	0.9	1.2
			5	3.5	169.5	0.6	1.1	1.6	1.8	166.4	0.4	0.6	1.3	1.5	163.4	0.2	0.5	0.9	1.3
			7	3.0	219.5	0.6	1.1	1.9	2.3	214.8	0.3	0.7	1.3	1.9	191.1	0.2	0.5	1.0	1.7
			9	2.5	268.0	0.5	1.1	2.0	2.7	261.5	0.2	0.7	1.3	2.2	203.1	0.3	0.7	1.2	1.8
1/2	#3	2x4	4	3.5	84.0	0.4	0.7	1.1	1.5	82.6	0.3	0.5	0.8	1.2	81.3	0.2	0.4	0.6	0.9
			5	3.0	109.0	0.4	0.8	1.2	1.7	107.0	0.3	0.5	0.9	1.3	105.0	0.1	0.3	0.6	1.0
			7	2.5	141.1	0.4	0.8	1.3	1.9	138.1	0.2	0.5	0.9	1.4	135.1	0.1	0.3	0.6	1.0
			9	2.5	172.3	0.4	0.8	1.3	2.0	168.1	0.2	0.5	0.9	1.5	148.5	0.1	0.4	0.7	1.2
3/4	#1	2x4	4	3.5	152.3	0.8	1.2	1.3	1.5	150.0	0.5	1.0	1.1	1.3	147.9	0.4	0.7	0.9	1.1
			5	3.5	178.5	0.8	1.5	1.8	2.0	175.5	0.5	1.0	1.5	1.7	172.7	0.3	0.7	1.2	1.5
			7	3.0	229.3	0.8	1.5	2.3	2.7	224.8	0.5	1.0	1.7	2.3	220.5	0.3	0.6	1.2	1.9
			9	2.5	278.5	0.8	1.5	2.4	3.2	272.2	0.4	0.9	1.7	2.6	264.2	0.2	0.6	1.2	2.0
3/4	#3	2x4	4	3.5	97.9	0.5	0.9	1.4	1.5	96.4	0.4	0.7	1.1	1.3	95.1	0.2	0.5	0.8	1.1
			5	3.5	114.7	0.5	1.0	1.5	2.0	112.9	0.4	0.7	1.1	1.6	111.0	0.2	0.5	0.8	1.2
			7	3.0	147.4	0.5	1.0	1.5	2.2	144.5	0.3	0.7	1.1	1.6	141.7	0.2	0.5	0.8	1.2
			9	2.5	179.0	0.5	1.0	1.6	2.3	175.0	0.3	0.7	1.1	1.7	171.2	0.2	0.4	0.8	1.3
1-1/8	#1	2x4	4	3.5	170.1	1.2	1.4	1.5	1.6	168.0	0.8	1.1	1.2	1.4	166.0	0.6	0.9	1.1	1.2
			5	3.5	197.2	1.2	1.8	2.0	2.1	194.5	0.8	1.5	1.7	1.8	191.9	0.6	1.1	1.4	1.6
			7	4.0	249.8	1.2	2.2	3.0	3.2	245.7	0.8	1.5	2.4	2.8	241.8	0.6	1.1	1.8	2.4
			9	3.5	300.8	1.2	2.2	3.4	4.0	295.1	0.8	1.5	2.4	3.4	289.6	0.5	1.1	1.8	2.7

Table A2-2B (Concluded)

Panel Width: 48 in.

Top & Bottom Skins		Stringers		Req'd* Bearing Lenght Ea. End	Panel Height = 7 ft					Panel Height = 8 ft					Panel Height = 9 ft				
					P_{da}# (kips)	Pdm(psi) for P_d/P_{da} =				P_{da}# (kips)	Pdm(psi) for P_d/P_{da} =				P_{da}# (kips)	Pdm(psi) for P_d/P_{da} =			
Nom. Thick.	Face Ply	Nom. Size	No.			0.8	0.6	0.4	0.2		0.8	0.6	0.4	0.2		0.8	0.6	0.4	0.2
1/2	#1	2x6	4	6.0	201.2	1.1	2.1	2.5	2.7	199.4	0.8	1.5	2.1	2.3	197.6	0.6	1.1	1.8	2.0
			5	5.5	266.7	1.2	2.3	3.0	3.2	264.0	0.8	1.7	2.5	2.8	261.3	0.6	1.2	[illegible]	2.4
			7	4.5	356.2	1.3	2.5	3.8	4.2	352.0	0.9	1.8	2.9	3.6	347.8	0.6	1.2	[illegible]	3.1
			9	4.5	444.6	1.4	2.8	4.4	5.2	438.7	0.9	1.9	3.1	4.4	433.0	0.6	1.3	[illegible]	3.6
1/2	#3	2x6	4	5.5	129.4	0.7	1.3	2.0	2.6	128.2	0.5	1.0	1.5	2.1	127.0	0.4	0.7	1.2	1.6
			5	5.0	171.4	0.8	1.5	2.4	3.1	169.7	0.6	1.1	1.7	2.5	168.0	0.4	0.8	1.3	1.9
			7	4.5	229.0	0.9	1.7	2.6	3.7	226.3	0.6	1.2	1.9	2.8	223.6	0.4	0.9	1.4	2.1
			9	4.0	285.8	1.0	1.9	2.9	4.1	282.0	0.7	1.3	2.1	3.1	278.4	0.4	0.9	1.6	2.4
3/4	#1	2x6	4	5.5	229.7	1.4	2.2	2.3	2.5	227.8	1.0	1.8	2.0	2.1	225.9	0.7	1.4	1.7	1.9
			5	5.5	275.3	1.4	2.7	3.2	3.4	272.8	1.0	2.0	2.7	3.0	270.3	0.7	1.4	2.3	2.6
			7	5.0	365.3	1.6	2.9	4.2	4.6	361.3	1.1	2.1	3.3	3.9	357.3	0.7	1.5	2.5	[illegible]
			9	4.5	454.1	1.7	3.1	4.9	5.6	448.4	1.1	2.2	3.6	4.8	442.9	0.7	1.6	2.6	[illegible]
3/4	#3	2x6	4	6.0	147.7	0.9	1.7	2.5	2.6	146.5	0.7	1.3	1.9	2.3	145.3	0.5	0.9	1.5	2.0
			5	5.5	177.0	1.0	1.8	2.7	3.4	175.4	0.7	1.3	2.0	2.8	173.7	0.5	1.0	1.5	2.2
			7	4.5	234.9	1.1	1.9	3.0	4.1	232.3	0.7	1.4	2.2	3.1	229.7	0.5	1.0	1.7	[illegible]
			9	4.5	291.9	1.1	2.1	3.2	4.5	288.3	0.8	1.5	2.4	3.4	284.7	0.5	1.1	1.8	[illegible]
1-1/8	#1	2x6	4	5.5	247.3	1.9	2.3	2.4	2.5	245.6	1.4	1.9	2.0	2.2	243.9	1.0	1.6	1.8	1.9
			5	5.5	293.6	1.9	3.1	3.2	3.4	291.3	1.4	2.6	2.8	2.9	289.0	1.1	2.0	2.4	2.6
			7	5.5	384.8	2.1	3.8	4.9	5.2	381.2	1.5	2.8	4.1	4.5	377.6	1.1	2.1	3.3	3.9
			9	5.0	474.6	2.2	4.0	5.9	6.4	469.5	1.5	2.9	4.5	5.5	464.4	1.1	2.1	3.4	4.7
1-1/8	#1	2x8	4	7.5	313.7	2.6	3.2	3.3	3.4	312.2	1.9	2.7	2.8	2.9	310.7	1.5	2.3	2.5	2.6
			5	7.5	376.7	2.7	4.4	4.5	4.7	374.6	2.0	3.7	3.9	4.1	372.6	1.5	2.9	3.4	3.6
			7	7.5	501.5	3.0	5.6	6.8	7.1	498.3	2.2	4.2	5.8	6.2	495.1	1.6	3.2	4.9	5.4
			9	7.0	625.3	3.2	6.1	8.5	8.9	620.7	2.4	4.5	6.9	7.7	616.2	1.7	3.4	5.3	6.7

PSSP Upgrading Example - Column

Try one of the PSSP designs as an added column in upgrading a basement for shelter. Assume a closed shelter, therefore no blast or other lateral load on the PSSP.

Assume use of a PSSP design on p. A2-18, that is, the one with ½" th. #3 top and bottom skins with 4 lower strength 2x4 stringers: Table A2-2A shows a max. column load P_{da} of 84 kips, which includes an increase of normal use design by 4x and use of $\mu = 1$ (see p. A2-7, 4th paragraph).

Thus, for example, the 84^k column, supporting a floor contributory area of 5' by 6', or 30 sf, will take a blast loading on the floor over the basement of:

$$p_s = \frac{84000}{30} \times \frac{1}{144} = 19 \text{ psi}$$

A more precise figure, used below for demonstration and not to imply a degree of accuracy, would be:

$$p_s = 19.44 \text{ psi (blast loading)}$$

If soil loads are added on the floor for fallout shielding, say 200 psf:

$$P_{(static\ loads)} = P_s \times \frac{1}{4} \times 2 = \frac{19.44}{4} \times 2 = 9.72 \text{ psi}$$

where ¼ represents removing the multiple of 4 taken (in the table) for blast loads, and 2 represents removing the effect of $\mu = 1$ in preparing the table.

The fallout soil, if left on the floor for less than 2 mos. merits a 15% increase in allowable stress, thus, the remaining static load capacity

$$P_{(static\ loads)} = 9.72 - \left[\frac{200}{1.15} \times \frac{1}{144}\right]$$
$$= 9.72 - 1.21 = 8.51 \text{ psi}$$

Reconverted to blast resistance capacity (but with 200 psf soil added to the floor load):

$$P_s = 8.51 \times 4 \times \frac{1}{2} = 17 \text{ psi (blast loading)}$$

NOTATION

A = total x-sectional area of longitudinal grain material in both plywood skins and stringers (in.^{2})

b = least dimension of solid rectangular cross-section (in.)

c = distance from N.A. (bending) to extreme fiber in compression(in.)

d = greater dimension of solid rectangular cross-sectio (in.)

d' = depth of PSSP stringers (in.)

E = modulus of elasticity (psi)

(EI_g) = stiffness factor for moment deflection [1(Sec.2.4.3)] (lbs-in.^{2} for full panel)(from Step 4, Appendix A1)

(EI_n) = stiffness factor for bending moment [1(Sec.2.5.3)] Ilb-in.^{2} for full panel) (from Step 8, Appendix A1)

F_c = allowable compressive stress (parallel to grain) for plywood skins (psi) [2(p.17)], corrected for buckling [1(Sec.2.5.4)]

I_g = gross moment of inertia of cross-section (in.^{4})

I_n = bending moment of intertia of full panel (in.^{4})

ℓ = clear span of member (simply-supported/pin-ended)(in.)

ℓ' = width of PSSP skins (perpendicular to stringers) (in.)

M = allowable bending moment (in.-lb), under combined loading

M_{max} = maximum moment caused by transverse loads only (in.-lb)

P = allowable axial load (lbs), under combined loading

P_a = allowable axial load (lbs), if axial load only exists

P_d = dynamic (blast) load value related to P (lbs)

P_{da} = dynamic (blast) load value related to P_a (lbs)

p_m = smallest of calculated allowable transverse (only) loads (in PSSPs, calculated loads for: deflection, bending moment, rolling shear and horizontal shear) (psi) (from Appendix A1 design of PSSPs)

p_{dm} = dynamic (blast) load value related to p_m (psi)

NOTATION (concluded)

p'_m = an intermediate value between p_m and zero (psi)

p'_{dm} = dynamic (blast) load value corresponding to p'_m (psi)

r = radius of gyration (in.)

S = section modulus of full panel (in.^{3})

x = location being examined (length along member) (in.)

y = deflection of column at M (in.)

$\bar{y}$ = deflection of column at M_{max} (transverse loads only (in.)

μ = ductility ratio = x_m / x_e (see Appendix A)

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9. Newmark, N. M., Design of Openings for Buried Shelters, Report 2-67, U.S. Army Engineer Waterways Experiment Station, Vicksburg, Mississippi, July 1963; p. 151-182.

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[10] Now SRI International

[11] Now Federal Emergency Management Agency

Appendix A3

PLYWOOD USE FOR CLOSURES - DESIGN

Extracts (with minor revisions) from the main text and Appendix A1 of

Murphy, H. L., Upgrading Basements for Combined Nuclear Weapons Effects: Predesigned Expedient Options, Stanford Research Institute* Technical Report, for U.S. Defense Civil Preparedness Agency,# October 1977. (AD-A054 409)

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* Now SRI International

\# Now Federal Emergency Management Agency

CONTENTS

TABLES

Background

Appendices A1 and A2, in their early paragraphs, describe plywood uses toward meeting the need for expedient aperture closures and added overhead floor system supports, respectively, in the upgrading of existing basements for shelter use against the combined effects of a nuclear weapons detonation. This Appendix A3 closes the area of plywood applications by describing a design approach for simple use of plywood for closures, especially over those many shelter openings having a rather short span in at least one of its two directions.

Approach

Use was made of two publications and telephone discussions[1-3] in developing a design procedure for use of plywood to close apertures in existing basements. The tables of the simplified publication[2] could not be reproduced through use of the design manual[1] procedures; requested clarification brought the recommendation that the latter be used for the purposes contemplated herein.[3]

As before, in Appendices A1 and A2, design formulas[1(pp.22-3,Sec.4)] were converted to the Notation herein and made dimensionally consistent. The revised formulas follow; plywood weight is ignored as dead load, and single spans, uniform loads, and simple supports are assumed.

The user is cautioned to apply care in units used in entering all values in the equations below; all equations are dimensionally consistent, i.e., there are no units hidden in the constants.

A. For uniform loads based on allowable <u>bending stress</u>:

$$p_b = 8\ F_b S\ /\ \ell^2 \qquad \text{(Eq.A3-1)}\dagger$$

where*

p_b = allowable load - bending moment (psi)

F_b = allowable bending stress (psi)

---

* Variables are defined herein at point of first use and in Notation at end of appendix.

† In Eq. A3-1, clear span can be used (per Reference 3 fonecon of 1/6/78).

S = effective section modulus (in.^{3}/in.width)

ℓ = clear span (in.)

B. For uniform loads based on allowable rolling shear stress:

$$p_s = 2\ F_s\ (Ib/Q)\ /\ \ell \qquad \text{(Eq.A3-2)}$$

where

p_s = allowable load - rolling shear stress (psi)

F_s = allowable rolling shear stress (psi)

(Ib/Q) = rolling shear constant (in.^{2}/in. width)

ℓ = clear span (in.)

The useful allowable load p_m then becomes:

$p_m = p_b$ or p_s whichever is smaller (psi) (Eq.A3-3)

C. For bending deflection (elastic) under uniform load:

$$y_b = p_m \ell^4\ /\ (76.8\ I\ (1.1\ E)) \qquad \text{(Eq.A3-4)*}$$

where

y_b = bending deflection (elastic) under uniform load (in.)

I = effective moment of inertia (in.^{4}/in. width)

E = modulus of elasticity (psi)

D. For shear deflection (elastic) under uniform load:

$$y_s = p_m C t^2 \ell^2\ /\ (106\ EI) \qquad \text{(Eq.A3-5)}$$

where

y_s = shear deflection (elastic) under uniform load (in.)

C = 120 or 60, for panels applied with face grain perpendicular to or parallel to supports, respectively.

t = nominal panel thickness (in.)

E. For combined bending and shear deflection (elastic) under uniform load: either (a) add y_b and y_s from Equations 4 and 5; or (b) use Equation 4 only, but with the constant 1.1 dropped from the equation.

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* Modified very slightly as to ℓ from Ref. 1, for simplification and because of negligible effect on the uses made of deflection calculations herein.

F. For plywood face bearing under uniform load (at ends over simple supports):

$$\ell_e = \ell \,/\, (2(F_{c\perp}/p_m - 1)) \qquad \text{(Eq.A3-6)}$$

where

ℓ_e = required plywood (face) end bearing length at each end of panel (in.)

$F_{c\perp}$ = allowable bearing stress on plywood face, for load perpendicular to plane of outer ply actually in bearing (psi)

It is recommended that ℓ_e be at least 1.5 in.

Design Procedure

The suggested design procedure consists of the following Steps:

1. Assume use of a particular plywood type, grade, nominal thickness t, and face ply(ies) species group (pp. 9, 14 and 15)† except that the latter must not be #5. Also assume that panel is uniformly loaded and simply supported,‡ and assume value for span ℓ (in.). Neglect the plywood weight as a DL.

2. Determine values (p. 16)† for I, S (=KS), and (Ib/Q), taking care to correct the units to in.^{4}, in.^{3}, and in.^{2} (all per in. width), respectively. Take care to use proper values for plywood used with the face grain running parallel to the span (cols. 5-7)† or perpendicular to the span (col. 9-11),† as well as the appropriate table (1 or 2)† and section (Unsanded, Sanded, or Touch-Sanded Panels).† If permitted by available supplies, plywood panels are used with the face grain running parallel to the span, which takes advantage of the stronger direction of the plywood.

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† See Reference 1; it is necessary that the designer hold this reference.
‡ On two opposite sides; but Step 7 extends the procedure to plywood panels supported on four sides.

3. Study the plywood data (p.14)* and select appropriate use condition (Wet or Dry) and grade stress level (S-1, -2, or -3). Determine values (p. 17)* for F_b, F_s, $F_{c\perp}$, and E (all psi).

4. Solve Equations 1-3 for p_b, p_s, and p_m, respectively.

5. Solve Equation 6 for ℓ_e.

6. If deflections are needed or desired, either:

(a) Solve Equations 4 and 5 for y_b and y_s, respectively ; then $y = y_b + y_s$; or

(b) Solve Equation 4 with the value 1.1 deleted on right side; then $y = y_b$.

7. For plywood panels supported on four sides, the procedure is as follows:[2,3]

(a) Complete Steps 1-4 and 6 for each span direction, finding p_m and y for each direction;

(b) Reduce the p_m value associated with the larger y, by multiplying that p_m by the ratio of the smaller y to the larger y. The two y values will then be equal, and the total capacity p_m of the panel supported on four sides will be the sum of the p_m just reduced and the unchanged p_m associated with the smaller y of Step 7a; use the latter two p_m values to find ℓ_e in each direction (Step 5).

Design Stresses - Blast Protection Use versus Normal Use

An Appendix A1 section with the same title applies fully herein, excepting that $\mu = 3$ is recommended for this appendix; thus, $p_{dm} = (5/6)\, p_m$, and F_b and F_s (but not $F_{c\perp}$ and E) are multiplied by four.

Typical Designs of Plywood Panels as Closures

Data in the preceding sections have been used to prepare the typical designs of plywood panels as closures shown in Tables A3-1A, -1B, -1C and -2. Computer programs used are listed in Table A3-3 of the original publication but not extracted herein.

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* See Reference 1; it is necessary that the designer hold this reference.

Table A3-1A PLYWOOD PANELS AS CLOSURES (ONE-WAY)

Plywood panels considered herein are each stamped with American Plywood Association (APA) Type (Interior or Exterior), Grade and, in most cases, with Face Ply Species Group(s) (the latter exception is discussed further below), as follows:

Plywood Type and Grade	Table A3-1B&C Block Nos.
C-D INTERIOR (APA),* usual:	3,11
If "interior with exterior glue" is specified:	2,10
UNDERLAYMENT INTERIOR (APA), usual:	8,16
If "interior with exterior glue" is specified:	7,15
C-D PLUGGED INTERIOR (APA), usual:	8,16
If "interior with exterior glue" is specified:	7,15
2.4.1 STURD-I-FLOOR INTERIOR (APA), with veneer inner plies only	17
APPEARANCE GRADES (Interior) (APA),† usual:	6,14
If "interior with exterior glue" is specified:	5,13
C-C EXTERIOR (APA)*	1,9
UNDERLAYMENT EXTERIOR (APA)	7,15
C-C PLUGGED EXTERIOR (APA)	7,15
2.4.1 STURD-I-FLOOR EXTERIOR (APA), with veneer inner plies only	17
APPEARANCE GRADES (Exterior) (APA),‡ with Surface A or C, face & back:	4,12
With Surface B face or back:	5,13

* Face Ply Species Groups are as follows: When stamped 24/0 on 1/2 in. (13 mm) thick plywood, Group 4; 32/16, Group 1; on 3/4 in. (19 mm): 42/20, Group 3; 48/24, Group 1.

† Generally applied where a high quality surface is required; includes N-N, N-A, N-B, N-D, A-A, A-B, A-D, B-B and B-D INTERIOR (APA) Grades.

‡ Generally applied where a high quality surface is required; includes A-A, A-B, A-C, B-B, B-C, HDO and MDO EXTERIOR (APA) Grades.

Table A3-1B PLYWOOD PANELS AS CLOSURES (ONE-WAY)*

Block No.	PLYWOOD Nom. Th. in.	Surface Finish	Grade Str. Level	Face Ply Grp	(SIDE-ON) PEAK AIR BLAST OVERPRESSURE VS. CL. SPAN — Clear Span, in. 4	6	8	10	12	14	16	18	20	22	24	26	28
1.	1/2	UNSANDED	S-1	1	31	21	15	11	8	6	psi						
				2,3	31	21	12	8	5								
				4	31	20	11	7	5								
2.	1/2	UNSANDED	S-2	1	31	21	14	9	6	5							
				2,3	31	18	10	7	5								
				4	31	17	10	6									
3.	1/2	UNSANDED	S-3	1	28	19	14	9	6	5							
				2,3	28	18	10	7	5								
				4	28	17	10	6									
4.	1/2	SANDED	S-1	1	36	24	18	12	8	6	5						
				2,3	36	23	13	8	6								
				4	36	22	12	8	5								
5.	1/2	SANDED	S-2	1	36	24	15	10	7	5							
				2,3	36	20	11	7	5								
				4	36	18	10	7	5								
6.	1/2	SANDED	S-3	1	32	21	15	10	7	5							
				2,3	32	20	11	7	5								
				4	32	18	10	7	5								
7.	1/2	TOUCH-S.	S-2	1	31	21	16	10	7	5							
				2,3	31	20	11	7	5								
				4	31	19	10	7	5								
8.	1/2	TOUCH-S.	S-3	1	28	19	14	10	7	5							
				2,3	28	19	11	7	5								
				4	28	19	10	7	5								
9.	3/4	UNSANDED	S-1	1	50	33	25	20	15	11	9	7	6	5			
				2,3	50	33	24	16	11	8	6	5					
				4	50	33	23	15	10	8	6	5					
10.	3/4	UNSANDED	S-2	1	50	33	25	18	13	9	7	6	5				
				2,3	50	33	21	13	9	7	5						
				4	50	33	19	12	9	6	5						
11.	3/4	UNSANDED	S-3	1	45	30	23	18	13	9	7	6	5				
				2,3	45	30	21	13	9	7	5						
				4	45	30	19	12	9	6	5						
12.	3/4	SANDED	S-1	1	58	39	29	20	14	10	8	6	5				
				2,3	58	39	22	14	10	7	5						
				4	58	37	21	13	9	7	5						
13.	3/4	SANDED	S-2	1	58	39	26	17	12	8	6	5					
				2,3	58	33	19	12	8	6	5						
				4	58	31	17	11	8	6							
14.	3/4	SANDED	S-3	1	53	35	26	17	12	8	6	5					
				2,3	53	33	19	12	8	6	5						
				4	53	31	17	11	8	6							
15.	3/4	TOUCH-S.	S-2	1	51	34	25	17	12	9	7	5					
				2,3	51	34	20	13	9	6	5						
				4	51	33	18	12	8	6	5						
16.	3/4	TOUCH-S.	S-3	1	46	31	23	17	12	9	7	5					
				2,3	46	31	20	13	9	6	5						
				4	46	31	18	12	8	6	5						
17.	1-1/8	TOUCH-S.	S-2	1-3		52	39	31	25	19	14	11	9	8	6	5	5

* Face ply grain running in span direction (i.e., perpendicular to the two supports). Required bearing length at each end (beyond clear span) is 1½ in. (38 mm) in all cases.

Table A3-2 PLYWOOD PANELS AS CLOSURES (TWO-WAY)

The purpose of this Table is to provide conversion percentages (increases) so that the user can use the data of Table A3-1B to obtain overpressure versus clear span data for two-way plywood panels (that is, supported on all four sides of the opening/aperture to be closed).

This Table is based on using plywood panels with their face ply grain running in the direction of the <u>shorter</u> of the aperture's two clear spans. Its results are expressed in terms of the ratio of the longer to the shorter of the two clear spans; such results are expressed as percentage increases in overpressure resistance values applied to the values in Table A3-1B, with such increases related to the BLOCK NUMBERS of the table.

Recommended support bearing length on all four sides is $1\frac{1}{2}$ in.

TABLES A3-1B&C BLOCK NUMBERS	RATIO OF LONGER TO SHORTER CLEAR SPANS 1:1	1.25:1	1.5:1	2:1
1 - 3	6%	2%	1%	*
4 - 6	23	10	5	1%
7, 8	7	3	1	*
9 - 11	15	6	3	1
12 - 14	47	19	9	3
15, 16	19	8	4	1
17	43	18	9	3

* Less than 1/2%

NOTATION

C — 120 or 60, for panels applied with face grain perpendicular to or parallel to supports, respectively

E — modulus of elasticity (psi)

F_b — allowable bending stress (psi)

$F_{c\perp}$ — allowable bearing stress on plywood face, for load perpendicular to plane of outer ply actually in bearing (psi)

F_s — allowable rolling shear stress (psi)

I — effective moment of inertia (in.^{4}/in. width)

(Ib/Q) — rolling shear constant (in.^{2}/in.width)

L — span center-to-center of supports (in.)

ℓ — clear span (in.)

ℓ_e — required plywood (face) end bearing length at each end of panel (in.)

p_b — allowable load - bending moment (psi)

p_{dm} — dynamic (blast) uniform load capacity (psi)

p_m — smaller of p_b or p_s = static uniform load capacity (psi)

p_s — allowable load - rolling shear stress (psi)

S — effective section modulus (in.^{3}/in. width)

t — nominal panel thickness (in.)

y — deflection (elastic) under uniform load (in.)

y_b — bending deflection (elastic) under uniform load (in.)

y_s — shear deflection (elastic) under uniform load (in.)

REFERENCES

1. Plywood Design Specifications (PDS), American Plywood Association, 1119 A Street, Tacoma, Washington 98401, Revised December 1976.*

2. Plywood Design Manual - Shelving, American Plywood Association, 1119 A Street, Tacoma, Washington 98401, 1975.

3. Personal communications: Author with Wm. A. Baker, P.E., Head, Engineering Service, Applied Research Service, American Plywood Association, 1119 A Street, Tacoma, Washington 98401, July 22, 1977; and with James Elliott of Mr. Baker's staff, August 1, 1977.

* An April 1978 revision is now available and can be used; it was checked to determine the need for any recalculations for this extract from the Appendix A3 as previously published.

Appendix B1

WOOD BEAM AND COLUMN DESIGN - SIMPLY SUPPORTED

Revision of Appendix B, Design of Wood Beams, Simply Supported, as published in:

Murphy, H. L., and J. E. Beck, Slanting for Combined Nuclear Weapons Effects: BLAST-RESISTANT DESIGN/ANALYSIS WITH EXAMPLES, Stanford Research Institute[1] Final Report, for Defense Civil Preparedness Agency,[2] December 1974. (AD-A016 631)

Murphy, H. L., J. R. Rempel, and J. E. Beck, SLANTING IN NEW BASEMENTS FOR COMBINED NUCLEAR WEAPONS EFFECTS: A Consolidated Printing of Four Technical Reports, 3 Vols., Stanford Research Institute[2] Technical Reports, for Defense Civil Preparedness Agency,[3] October 1975. (AD-A023 237)

Murphy, H. L., Upgrading Basements for Combined Nuclear Weapons Effects: Predesigned Expedient Options, Stanford Research Institute[2] Technical Report, for U.S. Defense Civil Preparedness Agency[3], October 1977. (AD-A054 409)

[1] Now SRI International

[2] Now Federal Emergency Management Agency

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TABLES

FIGURES

Appendix B1

WOOD BEAM AND COLUMN DESIGN - SIMPLY SUPPORTED

This appendix deals with solid wood used in beams and columns.[3]

Wood Beams - Simply Supported

The wood contemplated for use under the design procedures described herein is structural or stress-graded lumber, which has been carefully graded in accordance with the standard grading rules for the appropriate trade association. A complete list of such associations is available; see Reference [1][4] Supplement, page 19. It is urged that all lumber contemplated for shelter use - specifically, lumber in structural components or members whose stress-resisting capability is important to the survival of shelterees (in contrast to such things as a door cross-brace that simply holds together the structurally significant members) - be reinspected and regraded by even poorly qualified personnel using the appropriate association's grading rules.

Other items for the designer's general consideration are:

- The lack of homogeneity in wood members dictates that every effort be made to design wood structural members so that they interact in such a manner as to transfer load from a weaker, below-standard member to the better members. Examples are: really good blocking between floor joists; and use of tongue-and-groove planking as members used flat in a blast door. This is "repetitive-member use," see Reference [1] Supplement, Table 4A, 4th column.

- Only very tight knots should be accepted in a situation such as that of an unclad wood shelter blast door where an air blast loading could make a missile or bullet out of a knot that is even slightly loose.

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[3] Beam design chart solutions herein are for only single-span simply-supported (SS) beams; correction factors cover multi-span beams (section C, page B1-14). The design procedure also covers propped cantilever (PC) and fixed-fixed (FF) support conditions.

[4] Brackets are used herein to indicate sources in the References list at the end of this appendix.

- Metal cladding may be indicated for some situations where wood is used, such as exposure to fires (or where required by local building code), but not necessarily when concern is only about exposure to a nuclear thermal pulse (which may well char the door without setting it on fire; the latter is a difficult thing to do to a flat wood wall).

Because this appendix is intended for use by engineers and architects, some technical competence in the usual design of wood structural members is assumed [2,3], and only those design considerations peculiar to nuclear blast effects loading will be treated in some detail in this appendix.

Reading/study of Appendix B2 may serve to increase the reader/user's understanding of wood design and use in upgrading.

A. Design Procedure

Because wood beams are available in specific dimensions, the general design approach is to select a trial member depth (measured in the direction of the applied load) and width, then find the air blast peak overpressure it can resist; this overpressure is compared to the specified overpressure to be resisted. The resistance of the selected member is based on elasto-plastic behavior and associated stress resistances in flexure (bending), horizontal shear, and bearing on a support, which resistances are checked in that order. Specifically, the flexure and horizontal shear resistances are found, and then a new trial member is selected, repeating these steps until the lesser of the two resistances is found to be sufficient to meet the expected blast load. The required bearing area is then found directly.

It is recommended that the beam design procedures and graphs that follow be used <u>only</u> for L / d values equal to or greater than five (5), because of doubt that they apply to "deep beams" (L / d less than 5).

The <u>design steps</u> are as follows:

(1) A design air blast peak overpressure is specified, also whether its loading geometry will provide: a side-on overpressure (as in a wood door mounted flush with the earth's surface); a fully reflected overpressure (as in the front wall of a rectangular building); or a peak value of the average loading caused by a combination of side-on and drag pressure (as in the side-wall or roof of a rectangular building) [4,Sec.4.38]. Related variables, in the same order of loading geometries, look like this:

$$p_{dm} = p_{so} \text{ or } p_r \text{ or } [(p_{so} + C_d q)\ L/2U] \qquad (1)$$

where q is the dynamic (wind) blast pressure (unlike the q for structural resistance used in the remainder of this section) [4,Table 4.40].

(2) A trial size of wood beam (actual depth d, measured in direction of load, and thickness or width b) and kind of structural or stress-graded lumber are selected, then the grading association's design stresses are determined from their publications [1, Supplement, Table 4A].[5] Need for the latter may be limited to F_b (extreme fiber stress in bending), F_v (horizontal shear stress), and $F_{c\perp}$ (compression stress perendicular to grain, or bearing stress as used for beams only herein). For the short duration loadings furnished by nuclear air blast, dynamic values of the above three design stresses are recommended [5] as follows:

$$F_{db} = 4F_b \text{ ; } F_{dv} = 4F_v \text{ ; and } F_{dc\perp} = F_{c\perp}$$

Some grading rules allow increases in design stress values for such things as: repetitive-member uses [1,Supplement,Table 4A,fourth column]; and, members used flatwise [1,Supplement,page 20,paragraph 6].

(3) A design ductility ratio μ is selected (see discussion in earlier Appendix A, General Comments on Blast-Resistant Design/Analysis General Approach). A value of 3 is recommended [5], certainly as an upper limit, and with 1.3 or 2 even better [6].

(4) A short design procedure [5] omits use of any loading decay (i.e., uses instead an instantaneously applied long duration load, or step pulse), load-mass factors, modulus of elasticity, elasto-plastic resistance function per se, etc., all in favor of the following approach: A step pulse is assumed, which is reasonable particularly when large yield weapons and short wood beams (therefore having very short periods of natural vibration) are considered.[6] The other things ignored have been found to have little effect on the structural member selected for most applications; and needed parameters then have the following relationship:

$$p_{dm}/q = 1 - (1 / (2\mu))$$

where q is the ultimate resistance to blast loading of the wood beam. Using the recommended value of $\mu = 3$, the equation becomes: $p_{dm} = (5/6)\ q$.

(5) Span L (center-center of supports) and support conditions are known or assumed. Formulas are included herein for three beam support conditions: simply supported (SS); propped cantilever (PC); and both ends fixed (FF).

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[5] But if a Table B1-1 value for F_v is higher (than that given in Ref. [1]), use it.

[6] Alternatives to this use of a step pulse are chart solutions and the Newmark ß Method [7] or (better) the Modified Newmark ß Method (by J. E. Beck) [8,p.6-162].

Table B1-1

ALLOWABLE HORIZONTAL SHEAR VALUES F_v (psi)

	Maximum Moisture Content		
	Unseasoned	19 percent	15 percent
Aspen	85	90	95
Balsam Fir	85	95	95
Black Cottonwood	70	75	80
California Redwood	115	120	130
Coast Sitka Spruce	90	95	100
Coast Species	90	95	100
Douglas Fir–Larch	130	140	145
Douglas Fir–South	130	140	145
Eastern Hemlock–Tamarack	120	130	135
Eastern Spruce	95	105	110
Eastern White Pine	90	95	100
Eastern Woods	85	95	95
Engelmann Spruce/Alpine Fir	95	105	110
Hem–Fir	105	110	115
Idaho White Pine	95	100	105
Lodgepole Pine	95	105	110
Mountain Hemlock	130	140	150
Northern Aspen	90	95	100
Northern Pine	100	105	110
Northern Species	90	95	100
Northern White Cedar	85	95	100
Ponderosa Pine–Sugar Pine	100	105	110
Red Pine	100	110	115
Sitka Spruce	105	115	120
Southern Pine	125	135	145
Spruce–Pine–Fir	95	105	110
Western Cedar	100	105	110
Western Hemlock	125	135	145
Western White Pine	90	100	105
White Woods (Western Woods, West Coast Woods, Mixed Species)	95	100	105

Source: National Design Specification for Wood Construction, 1977 Edition, National Forest Products Association, 1619 Massachusetts Avenue, N.W., Washington, D. C. 20036; art. 3.4.4.2

Author Comments: Use column of above table that specifies "19 percent" moisture content. For wood species not shown, consult "Design Values for Wood Construction," 1980 Supplement to above Source.

(6) Flexural or bending resistance q_b (in terms of load/unit area) is calculated for the trial member:

$$M = wL^2c = q_b\, bL^2c = F_{db}\, S = F_{db}\, bd^2 / 6$$

$$q_b = F_{db}(d/L)^2 / (6c) = 2F_b(d/L)^2 / (3c) \qquad (2)$$

where c = 1/8 (SS) and (PC), 1/12 (FF).

(7) Horizontal shear resistance q_v (in terms of load/unit area) is also calculated for the trial member, with horizontal shear equal to vertical shear and taken at a distance d in from each end of the member: [2(p.4-12),5(p.161)]

$$V = w(L-2d)c' = q_v\, b(L-2d)c' = 2AF_{dv} / 3 = 2bdF_{dv} / 3$$

$$q_v = 2F_{dv}\, d / (3c'(L-2d)) = 8F_v\, d / (3c'(L-2d)) \qquad (3)$$

where c' = 1/2 (SS) and (FF), 5/8 (PC), the latter value being approximate but close enough for the purposes herein. NOTE: EQ. 2 AND 3 INCLUDE THE MULTIPLE 4 OF STEP 2.

(8) Wood beam resistance q is then equal to the lesser value between q_b and q_v; q is converted to peak air blast pressure by using:

$$p_{dm} = q\,(1 - (1 / (2\mu))) \qquad (4)$$

or, when the recommended value of $\mu = 3$ is used, $p_{dm} = (5/6)\, q$.

(9) If p_{dm} is less than the design air blast peak overpressure specified in the first step herein, a larger beam, or a different wood or grade having larger design stresses, must be tried. If p_{dm} is larger than the design overpressure, then it may be desirable to try a smaller beam, or a different wood or grade, in an effort toward closer design. In either case, a new trial member requires that the designer return to the second step and repeat the procedure to this point.

(10) Required bearing length L' at each end of the wood beam is calculated as follows:

$$V = qbLc' = F_{c\perp}\, bL'$$

$$L' = qLc' / F_{c\perp} \qquad (5)$$

where the values of c' are the same as in step 7 above. It is recommended that L' be at least 2 inches.

Another presentation of wood beam formulas is given in Appendix B2 (page B2-20).

Limitations on the above design procedure are stated in the two applications that follow.

B. Application to a Closure (Shelter Door) Design

An application of wood beam design occurs when low-cost blast doors must be designed for shelters, in new designs or existing structures. For an application in existing structures, particularly, a pre-design or chart approach was needed as follows:

- An estimate, calculated or judgmental, is made of the blast resistance of the wall adjacent to an aperture (door or window opening) for which a wood blast door is needed. The only designed structural element will be a wood beam, or series of wood beams side-by-side and preferably tonque-and-groove,[7] simply supported on the two sides of the door frame (that has been either strengthened or found adequate to take the load from the door onto the wall).

- Stress-graded wood of various kinds (species and grades) in standard thicknesses[8] (2, 3, 4 inches, nominal; 1.5, 2.5, 3.5 inches, actual) are checked for availability. See Table B1-2 for several popular woods and their allowable stresses.

a. The pre-design or chart approach developed for simplified handling of this problem is as follows, using solid, multiple wood beams, side-by-side:

(1) Obtain a copy of the industry association grading rules for each kind of wood contemplated for possible use; from this, make a tabulation (for each kind of wood and each thickness) of design stresses (psi) stated for use under normal loading for:

[7] If not t&g (tongue-and-groove), use a light plywood sheet covering on the blast side; if either t&g or plywood-covered, "repetitive-member use" in F_b design values is appropriate [1,Supplement,Table 4A,4th column].

[8] Thickness and width are the terms applied to the smaller and larger cross-section dimensions, respectively, in the industry [1,Supplement, page 20,paragraph 6]. Engineers use thickness and width the same way in columns, but in beams they use width and depth as the cross-section dimensions perpendicular and parallel to the direction of loading, respectively (for example, the width of a beam 1.5"x3.5" in cross-section would be 1.5" if used edgewise, 3.5" if used flatwise (to the load direction)).

Table B1-2

SELECTED DIMENSION LUMBER SPECIES, SIZES AND GRADES *
(CONSTRUCTION GRADES)

Size and Grade		Douglas Fir-Larch	Western Hemlock	Western Pines (Ponderosa, Lodgepole, Sugar, and Idaho White)	Southern Pine	Northern Pine
		Allowable Unit Stresses in Normal Use, psi †				
Structural Light Framing (2" to 4" th., 2" to 4" wide)						
Stud (2x4s only)	F_b	925	800	600	900	725
	F_v	95 †	90	70	90	70
	$F_{c\perp}$	385	280	190	405	280
	E (x 10^6)	1.5	1.3	1.2	1.4	1.1
	F_c	*600*	*550*	*425*	*575*	*475*
Light Framing (2" to 4" th., 4" wide)						
Construction	F_b	1200	1050	775	1150	950
	F_v	95	90	70	100	70
	$F_{c\perp}$	385	280	190	405	280
	E	1.5	1.3	1.2	1.4	1.1
	F_c	*1150*	*1050*	*775*	*1100*	*875*
Standard	F_b	675	600	425	675	525
	F_v	95	90	70	90	70
	$F_{c\perp}$	385	280	190	405	280
	E	1.5	1.3	1.2	1.4	1.1
	F_c	*925*	*850*	*650*	*900*	*725*
Joists and Planks (2" to 4" th., 5" and wider)						
No. 1	F_b	1750	1550	1100	1700	1400
	F_v	95	90	70	90	70
	$F_{c\perp}$	385	280	190	405	280
	E	1.8	1.6	1.4	1.7	1.4
	F_c	*1250*	*1150*	*875*	*1250*	*975*
No. 2	F_b	1450	1250	925	1400	1100
	F_v	95	90	70	90	70
	$F_{c\perp}$	385	280	190	405	280
	E	1.7	1.4	1.3	1.6	1.3
	F_c	*1050*	*975*	*725*	*1000*	*825*
No. 3	F_b	850	750	550	800	650
	F_v	95	90	70	90	70
	$F_{c\perp}$	385	280	190	405	280
	E	1.5	1.3	1.2	1.4	1.1
	F_c	*675*	*625*	*450*	*625*	*525*

* Table is for visually stress graded lumber only, used at 19% maximum moisture content.

† Notation used is: F_b for extreme fiber in bending (repetitive member use); F_c for compression parallel to grain; F_v for horizontal shear; $F_{c\perp}$ for compression perpendicular to grain; and E for modulus of elasticity. For dynamic uses in this report, F_b, F_c, and F_v stresses are multiplied by 2.6 to 4.

‡ F_c values in italics are for bending members only; not recommended for PSSP stringers. See SOURCE below, main report not supplement, art. [illegible].

SOURCE: "Design Values for Wood Construction," 1980 Supplement to the 19[illegible] Edition of National Design Specification for Wood Construction, National Forest Products Association, 1619 Massachusetts Avenue, N.W., Washington, D.C. 20036.

- Bending design stress (in extreme fiber), in repetitive-member use, F_b

- Horizontal shear design stress, F_v

- Compression perpendicular to grain design stress, $F_{c\perp}$

(2) Conversion of design stresses to dynamic values (step 2 above) is unnecessary hereunder; the chart p_{dm} includes this conversion and the chart is therefore entered directly with the design stresses for <u>normal</u> loading. Similarly included is the factor (5/6) for $p_{dm} = (5/6)\ q$ (per design steps 4 and 8 above). Increases for "flatwise" use are <u>not</u> included in the chart p_{dm} (see design step 2 above).

(3) For each wood and thickness, determine the blast resistance in terms of free-field overpressure:

Enter the table on the page facing Figure B1-1 and read values A and B, for each pair of values of span L (in.) and (actual, not nominal) beam depth d (in.).

Enter the left portion of Figure B1-1 with the A and B values as follows: Use A (at top of figure) with the "curves" for horizontal shear F_v values and read q_n (or p_{dm}) on left (ordinate) scale; use B (at bottom of figure) with the "curves" for bending stress F_b values and read q_n (or p_{dm}) on left scale; <u>use only the smaller value of q_n (or p_{dm}) read</u>!

(4) For each wood and thickness still of interest, determine the required bearing length <u>at each end</u> of the wood beam:

Use the $F_{c\perp}$ value found above and the p_{dm} value just found, and calculate the required bearing length L′ (in.) at <u>each</u> end of the beam, from the following modification of Eq. 5:

$$L' = 0.6\ p_{dm}\ L\ /\ F_{c\perp} \qquad (6)$$

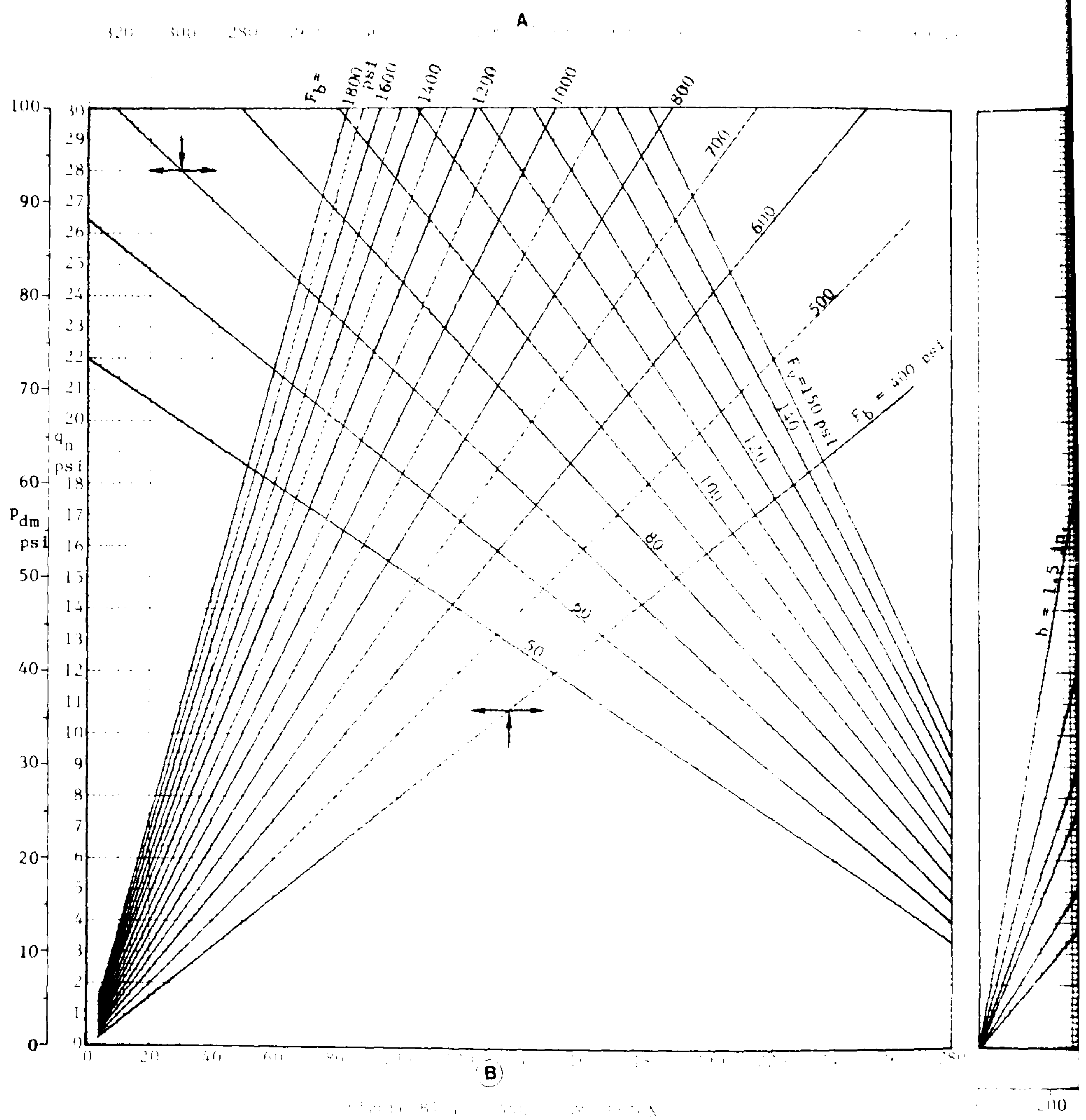
but not less than 2 in. Strictly speaking, if the beam-ends in bearing are exposed to the air blast, Eq. 6 should be:

$$L' = 0.6\ p_{dm}\ L\ /\ (F_{c\perp} - p_{dm}).$$

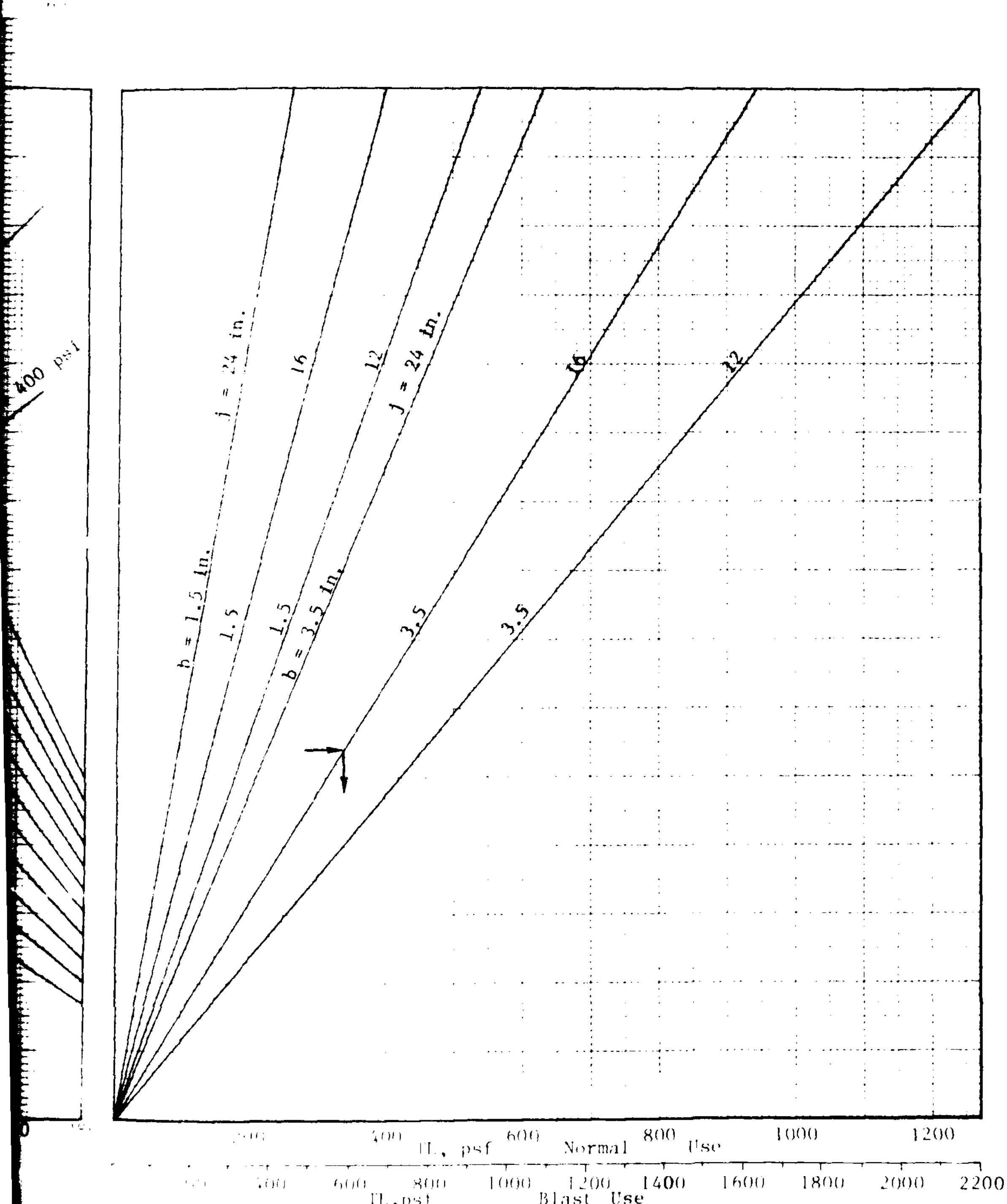
VALUES OF A AND B FOR FIGURE B1-1

Span	d=1.5 in.		=2.5 in.		=3.5 in.		=4.5 in.		=5.5 in.		=7.25 in.		=9.25 in.		=11.25 in.	
L (in.)	A	B	A	B	A	B	A	B	A	B	A	B	A	B	A	B
12	167	104	0	0	0	0	0	0	0	0	0	0	0	0	0	0
14	136	77	278	213	0	0	0	0	0	0	0	0	0	0	0	0
16	115	59	227	163	0	0	0	0	0	0	0	0	0	0	0	0
18	100	46	192	129	318	252	0	0	0	0	0	0	0	0	0	0
20	88	38	167	104	269	204	0	0	0	0	0	0	0	0	0	0
22	79	31	147	86	233	169	0	0	0	0	0	0	0	0	0	0
24	71	26	132	72	206	142	300	234	0	0	0	0	0	0	0	0
26	65	22	119	62	184	121	265	200	0	0	0	0	0	0	0	0
28	60	19	109	53	167	104	237	172	324	257	0	0	0	0	0	0
30	56	17	100	46	152	91	214	150	289	224	0	0	0	0	0	0
32	52	15	93	41	140	80	196	132	262	197	0	0	0	0	0	0
34	48	13	86	36	130	71	180	117	239	174	0	0	0	0	0	0
36	45	12	81	32	121	63	167	104	220	156	0	0	0	0	0	0
38	43	10	76	29	113	57	155	93	204	140	309	243	0	0	0	0
40	41	9	71	26	106	51	145	84	190	126	284	219	0	0	0	0
42	38	9	68	24	100	46	136	77	177	114	264	199	0	0	0	0
44	37	8	64	22	95	42	129	70	167	104	246	181	0	0	0	0
46	35	7	61	20	90	39	122	64	157	95	230	166	0	0	0	0
48	33	7	58	18	85	35	115	59	149	88	216	152	314	248	0	0
50	32	6	56	17	81	33	110	54	141	81	204	140	294	228	0	0
52	31	6	53	15	78	30	105	50	134	75	193	130	276	211	0	0
54	29	5	51	14	74	28	100	46	128	69	184	120	261	196	0	0
56	28	5	49	13	71	26	96	43	122	64	175	112	247	182	0	0
58	27	4	47	12	69	24	92	40	117	60	167	104	234	170	317	251
60	26	4	45	12	66	23	88	38	112	56	159	97	223	158	300	234
62	25	4	44	11	64	21	85	35	108	52	153	91	213	146	285	219
64	25	4	42	10	61	20	82	33	104	49	146	86	203	139	271	206
66	24	3	41	10	59	19	79	31	100	46	141	80	195	131	259	194
68	23	3	40	9	57	18	76	29	96	44	136	76	187	123	247	182
70	22	3	38	9	56	17	74	28	93	41	131	72	180	116	237	172
72	22	3	37	8	54	16	71	26	90	39	126	68	173	110	227	163
74	21	3	36	8	52	15	69	25	87	37	122	64	167	104	218	154
76	21	3	35	7	51	14	67	23	85	35	118	61	161	99	210	146
78	20	2	34	7	49	13	65	22	82	33	114	58	155	94	203	139
80	19	2	33	7	48	13	63	21	80	32	111	55	150	89	196	132
82	19	2	32	6	47	12	62	20	77	30	107	52	146	85	189	125
84	19	2	32	6	45	12	60	19	75	29	104	50	141	81	183	120
86	18	2	31	6	44	11	58	18	73	27	101	47	137	77	177	114
88	18	2	30	5	43	11	57	17	71	26	99	45	133	74	172	109
90	17	2	29	5	42	10	56	17	70	25	96	43	129	70	167	104
92	17	2	29	5	41	10	54	16	68	24	94	41	126	67	162	100
94	16	2	28	5	40	9	53	15	66	23	91	40	123	65	157	95
96	16	2	27	5	39	9	52	15	65	22	89	38	119	62	153	92
98	16	2	27	4	38	9	51	14	63	21	87	36	116	59	149	88
100	15	2	26	4	38	8	49	13	62	20	85	35	113	57	145	84
102	15	1	26	4	37	8	48	13	60	19	83	34	111	55	142	81
104	15	1	25	4	36	8	47	12	59	19	81	32	108	53	138	78
106	15	1	25	4	35	7	46	12	58	18	79	31	106	51	135	75
108	14	1	24	4	35	7	45	12	57	17	78	30	103	49	132	72
110	14	1	24	3	34	7	45	11	56	17	76	29	101	47	129	70
112	14	1	23	3	33	7	44	11	54	16	74	28	99	45	126	67
114	14	1	23	3	33	6	43	10	53	16	73	27	97	44	123	65
116	13	1	23	3	32	6	42	10	52	15	71	26	95	42	120	63
118	13	1	22	3	32	6	41	10	51	14	70	25	93	41	118	61
120	13	1	22	3	31	6	41	9	50	14	69	24	91	40	115	59
122	13	1	21	3	30	5	40	9	50	14	67	24	89	38	113	57
124	12	1	21	3	30	5	39	9	49	13	66	23	88	37	111	55
126	12	1	21	3	29	5	38	9	48	13	65	22	86	36	109	53
128	12	1	20	3	29	5	38	8	47	12	64	21	84	35	107	51
130	12	1	20	2	28	5	37	8	46	12	63	21	83	34	105	50
132	12	1	20	2	28	5	37	8	45	12	62	20	81	33	103	48
134	11	1	19	2	28	5	36	8	45	11	61	20	80	32	101	47
136	11	1	19	2	27	4	35	7	44	11	60	19	79	31	99	46
138	11	1	19	2	27	4	35	7	43	11	59	18	77	30	97	44
140	11	1	19	2	26	4	34	7	43	10	58	18	76	29	96	43
142	11	1	18	2	26	4	34	7	42	10	57	17	75	28	94	42
144	11	1	18	2	26	4	33	7	41	10	56	17	74	28	93	41

In above, $A = d / (L-2d) \times 1000$, and $B = (d/L)^2 \times (2/3) \times 10000$



A
F_b = 1800 psi
1600
1400
1200
1000
800
700
600
500
F_v = 150 psi
140
120
100
80
70
50
F_b = 400 psi
q_n psi
P_{dm} psi
B
b = 1.5 in.
200


400 psi
j = 24 in.
16
12
j = 24 in.
16
12
b = 1.5 in.
1.5
1.5
b = 3.5 in.
3.5
3.5
400
600
800
1000
1200
IL, psf Normal Use
600
800
1000
1200
1400
1600
1800
2000
2200
IL, psf Blast Use

(5) <u>As an illustrative example</u>: Using Douglas Fir-Larch 2x4s (on edge, solidly side-by-side), Structural Light Framing, Stud grade, read from Tabl B1-1: F_b (repetitive-member use) is 925 psi, F_v is 140 psi, and $F_{c\perp}$ is 385 psi. Span L is 40 in. center-center of supports. Enter table facing Figure B1-1 and read: A is 106, and B is 51. Enter Figure B1-1 with A and B and read: for A of 106 and F_v of 140, q_n is 19.8 psi; for B of 51 and F_b of 925, q_n is about 9.4 psi. Thus, the q_n of 9.4^+psi is the smaller, and p_{dm} of 31.5 psi is the peak air blast (free field) overpressure design resistance of this solid door of 2x4s on edge (assuming that the door frame has been checked and found adequate).[9]

Required bearing length on <u>each</u> end of each 2x4 is calculated using Eq. 6 and $F_{c\perp}$ of 385 minus p_{dm} (assuming the blast hits the supporting ends of the 2x4s, as it usually would), or an $F_{c\perp}$ of, say, 350 psi: $L' = 0.6 \times 31.5 \times 40 / 350 = 2\text{-}1/4$ in. bearing length on <u>each</u> end.

(6) The above example assumes that the closure is covered with plywood (e.g., 1/4 in.). <u>If not</u>: use F_b for single-member use [1,Table 4A,third column]; use a stress multiple of 2.6 (not 4),[10]; use a μ of 1.5 (with same step pulse); and, therefore, read q_n (not p_{dm}) when using Figure B1-1, and <u>calculate</u> $p_{dm} = 2.6\ (2/3)\ q_n = 1.73\ q_n$. Thus, the single-member use applied to the above illustrative example changes the resulting p_{dm} from 31.5 psi to $p_{dm} = 1.73\ q_n = 1.73 \times 9.4 = 16.3$ psi.

b. The chart approach is as follows <u>when using beams spaced as in floor stringers</u> (chart is only for beam widths b of 1.5 and 3.5 in., each spaced at 12, 16 and 24 in. center-center):

Using the same wood beams, span L (40 in.), F_v of 140 psi, A of 106, B of 51, and $F_{c\perp}$ of 385 psi as in the illustrative example above, find F_b (single-member use) of 800 psi [1,Supplement,Table 4A]; enter Figure B1-1 as before and read q_n values (q_n of 19.8 from A of 106 and F_v of 140; q_n of 8.2 from B of 51 and F_b of 800) of which 8.2 psi is the lower. The q_n value is stated only for illustration; the reader/user should move to the right from finding the intersection of B and F_b values - finding that the total load TL in pounds per <u>square foot</u> of covering (in normal use) over the spaced wood beams is: 150 psf TL for beam width b of 1.5 in. and spacing j of 12 in.; 113 psf for 16 in. spacing; and 75 psf for 24 in. spacing.

---

[9] The increase factor from q_n to p_{dm} is 3.333. It represents a 100% increase for short duration (impact) loading [1,sec.2.2.5.3], another 100% increase to use up the assumed factor of safety for repetitive-member use (reader/user may want to reduce this to 30% by hand calculation, if no regrading has been done), and a reduction by (5/6) for using $\mu = 3$ (reader/user may also want to reduce this by hand calculation): thus, the overall factor is $2 \times 2\ (5/6) = 3.333\ldots$, or $p_{dm} = 3.333\ q_n$.

[10] Representing increases of 100% for short duration (impact) loading and 30% for factor of safety in single-member use.

These normal-use design TL values can be corrected for air blast loading by use of a multiple of 1.73 (as in paragraph (6) just above), giving 260, 195, and 130 psf for spacings of 12, 16, and 24 in., respectively, as calculated, or such results may be read directly from the outer (top and bottom) scales of the right graph, Figure B1-1.

C. Support Conditions Other Than Single-Span Simply Supported (SS)

The foregoing deals with single-span beams, on simple (non-moment carrying) supports (SS). For two-equal-span beams on simple supports (PC), formulas for one span (of the two) are on pages B2-20 and -21, Appendix B2, using constants for PC. (The formulas for FF conditions are of little interest to the upgrading sizing herein.)

If the contemplated wood beam use involves one beam extending over more than a single span: find q_{nv} and q_{nb} for a single-span beam as before (i.e., from A-F_v and B-F_b, respectively); find related p_{dmv} and p_{dmb}; and find related TL_v and TL_b. Correct these values for multiple-span beams (with all simple supports) as follows [9,p.5-41]:

	2-span	3-span	4-span	5-span
q_n, p_{dm}, and TL with subscript b:	1.000	1.250	1.168	1.190
q_n, p_{dm}, and TL with subscript v:	0.800	0.833	0.824	0.836

Use the lower value of each: q_n, p_{dm}, and TL.

Wood Columns - Simple Supports

The wood columsn contemplated in this design section have square ends and receive/deliver loads through well-fitted joints (not fixed). (Such conditions are sometimes called pin-ended.) The columns may or may not have intermediate supports.

Much of the guidance about wood and the wood beam design approach (both as presented in the preceding section on wood beams) apply to column design. Accordingly, the column design procedure that follows is considerably abbreviated, as compared to the beam design section, and depends a lot on an illustrative example.

A. Design Procedure

a. Column (normal-use) design formulas are shown in Table B1-3. Note the sketch and use of two different L / d ratios where there is an intermediate support for the weak direction; per Eq. 7, only the larger L / d value is used further. Use the table facing Fig. B1-2A to get each needed L / d value; enter the table with span L and depth d (both d_1 and d_2 of the sketch in Table B1-3 are termed "depth" - call them larger and smaller d's, and similar for the L's); read the L / d value. The sketch of Table B1-3 actually shows a part-column with two sets of intermediate supports; if only the weaker direction has intermediate support, ℓ_2 is the end-to-end column length. If the (larger) design L / d value is greater than 50, the selected member and/or its lateral supports is/are inadequate; change member or supports.

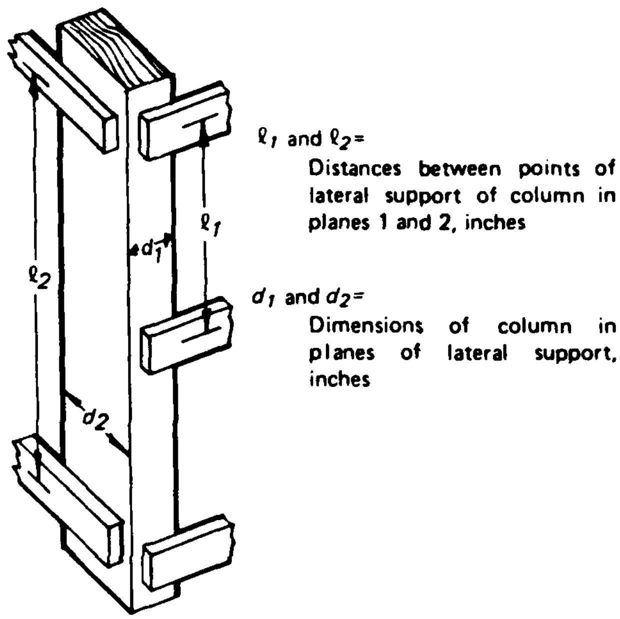
Determine the modulus of elasticity E and the design value for compression stress parallel to grain F_c for the wood member selected (1,Supplement,Table 4A); regrading is urged (time permitting), just as it was for wood beams.

Using Figure B1-2A, enter graph 1 with the F_c value; go up to the appropriate E value (or interpolate vertically between the two curves bracketing the E value), then right to read a value for K (on scale between graphs 1 and 2). If the design L / d value is less than or equal to K, continue to the right until hitting the appropriate L / d curve (or interpolate horizontally between L / d curves); using the same F_c (as used with graph 1) go vertically to the F_c line of graph 3 (or interpolate vertically); then go horizontally to the graph 4 line representing the nominal (actual) column dimensions; finally, go vertically to read P and P_d.

The final P found comes from use of Eq. 13. For P_d, multiples of 2 on F_c for short duration (impact) loading, and 1.3 for a 30% factor of safety, are reduced by a factor of 0.5 from using a μ of 1; thus P_d equals (2 x 1.3 x 0.5) or 1.3 times P.

Table B1-3

COLUMN FORMULAS - SIMPLE SOLID COLUMN DESIGN [3]
(Pin-ended conditions assumed)



For conditions shown in the sketch:

Use <u>larger</u> of L_1 / d_1 and L_2 / d_2 as <u>the</u> L / d for design (7)

Slenderness ratio L / d must be ≤ 50 (8)

<u>For $L / d \leq 11$</u>: $F'_c = F_c$ (9)

<u>For $L / d > 11$, but $\leq K$</u>:

$$K = 0.671 \sqrt{E / F_c} \tag{10}$$

$$F'_c = F_c \left(1 - 1/3 \left(L / dK\right)^4\right) \tag{11}$$

<u>For $L / d \geq K$</u>: $F'_c = 0.3E / (L / d)^2$ (12)

$$P = AF'_c = bdF'_c \tag{13}$$

L / d VALUES FOR FIGURE B1-2

L (in.)	d (in.)							
	1.5	2.5	3.5	4.5	5.5	7.25	9.25	11.25
18	12.0	7.2	5.1	4.0	3.3	2.5	1.9	1.6
20	13.3	8.0	5.7	4.4	3.6	2.8	2.2	1.8
22	14.7	8.8	6.3	4.9	4.0	3.0	2.4	2.0
24	16.0	9.6	6.9	5.3	4.4	3.3	2.6	2.1
26	17.3	10.4	7.4	5.8	4.7	3.6	2.8	2.3
28	18.7	11.2	8.0	6.2	5.1	3.9	3.0	2.5
30	20.0	12.0	8.6	6.7	5.5	4.1	3.2	2.7
32	21.3	12.8	9.1	7.1	5.8	4.4	3.5	2.8
34	22.7	13.6	9.7	7.6	6.2	4.7	3.7	3.0
36	24.0	14.4	10.3	8.0	6.5	5.0	3.9	3.2
38	25.3	15.2	10.9	8.4	6.9	5.2	4.1	3.4
40	26.7	16.0	11.4	8.9	7.3	5.5	4.3	3.6
42	28.0	16.8	12.0	9.3	7.6	5.8	4.5	3.7
44	29.3	17.6	12.6	9.8	8.0	6.1	4.8	3.9
46	30.7	18.4	13.1	10.2	8.4	6.3	5.0	4.1
48	32.0	19.2	13.7	10.7	8.7	6.6	5.2	4.3
50	33.3	20.0	14.3	11.1	9.1	6.9	5.4	4.4
52	34.7	20.8	14.9	11.6	9.5	7.2	5.6	4.6
54	36.0	21.6	15.4	12.0	9.8	7.4	5.8	4.8
56	37.3	22.4	16.0	12.4	10.2	7.7	6.1	5.0
58	38.7	23.2	16.6	12.9	10.5	8.0	6.3	5.2
60	40.0	24.0	17.1	13.3	10.9	8.3	6.5	5.3
62	41.3	24.8	17.7	13.8	11.3	8.6	6.7	5.5
64	42.7	25.6	18.3	14.2	11.6	8.8	6.9	5.7
66	44.0	26.4	18.9	14.7	12.0	9.1	7.1	5.9
68	45.3	27.2	19.4	15.1	12.4	9.4	7.4	6.0
70	46.7	28.0	20.0	15.6	12.7	9.7	7.6	6.2
72	48.0	28.8	20.6	16.0	13.1	9.9	7.8	6.4
74	49.3	29.6	21.1	16.4	13.5	10.2	8.0	6.6
76	50.7	30.4	21.7	16.9	13.8	10.5	8.2	6.8
78	52.0	31.2	22.3	17.3	14.2	10.8	8.4	6.9
80	53.3	32.0	22.9	17.8	14.5	11.0	8.6	7.1
82	54.7	32.8	23.4	18.2	14.9	11.3	8.9	7.3
84	56.0	33.6	24.0	18.7	15.3	11.6	9.1	7.5
86	57.3	34.4	24.6	19.1	15.6	11.9	9.3	7.6
88	58.7	35.2	25.1	19.6	16.0	12.1	9.5	7.8
90	60.0	36.0	25.7	20.0	16.4	12.4	9.7	8.0
92	61.3	36.8	26.3	20.4	16.7	12.7	9.9	8.2
94	62.7	37.6	26.9	20.9	17.1	13.0	10.2	8.4
96	64.0	38.4	27.4	21.3	17.5	13.2	10.4	8.5
98	65.3	39.2	28.0	21.8	17.8	13.5	10.6	8.7
100	66.7	40.0	28.6	22.2	18.2	13.8	10.8	8.9
102	68.0	40.8	29.1	22.7	18.5	14.1	11.0	9.1
104	69.3	41.6	29.7	23.1	18.9	14.3	11.2	9.2
106	70.7	42.4	30.3	23.6	19.3	14.6	11.5	9.4
108	72.0	43.2	30.9	24.0	19.6	14.9	11.7	9.6
110	73.3	44.0	31.4	24.4	20.0	15.2	11.9	9.8

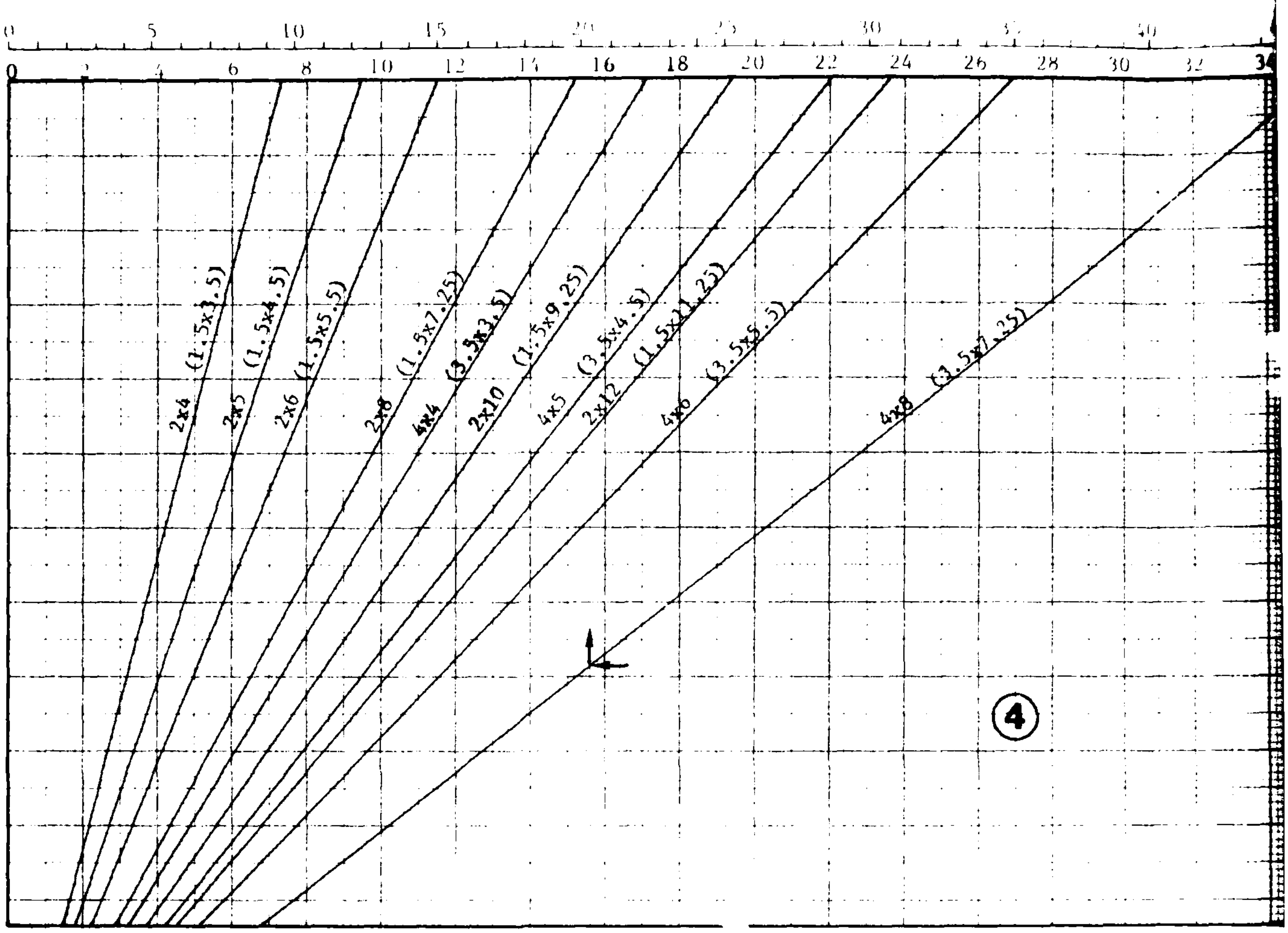
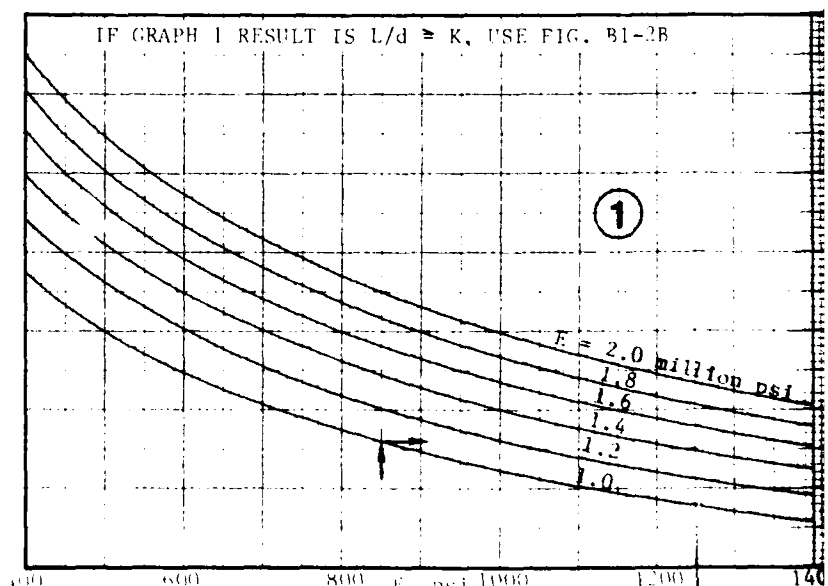
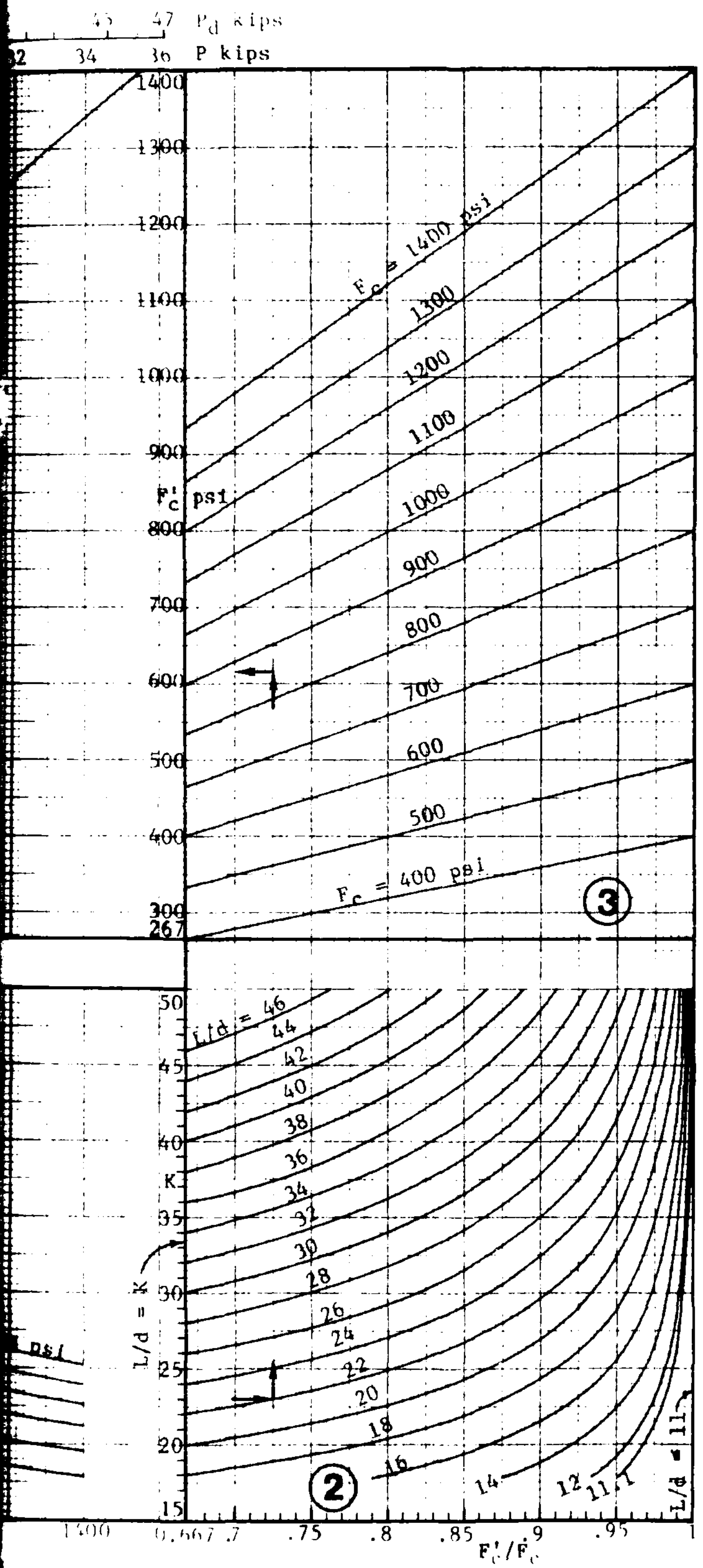


Figure B1-2A

COLUMN DESIGN, WOOD





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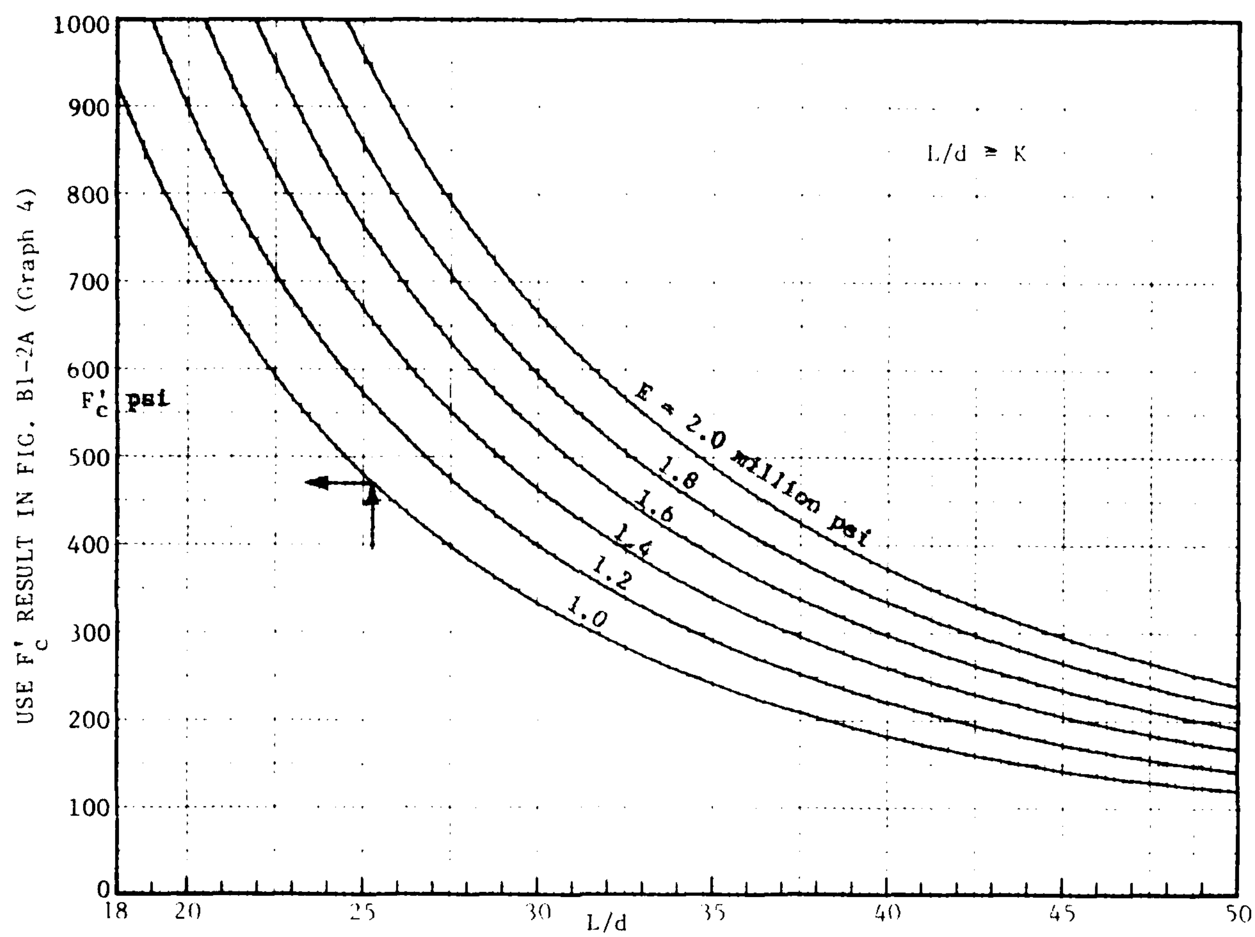

USE F'c RESULT IN FIG. B1-2A (Graph 4)
F'c psi
1000
900
800
700
600
500
400
300
200
100
0
18
20
25
30
35
40
45
50
L/d
L/d ≥ K
E = 2.0 million psi
1.8
1.6
1.4
1.2
1.0

Figure B1-2B

b. Use of the 1.5 multiple is recommended only if regrading is done. _If not:_ it is recommended that P_d be taken as equal to P, thus leaving intact the 30% factor of safety to cover grading errors and/or buckling hazards.

See page B2-29, Appendix B2, for a calculations example of handling combined (fallout shielding) soil loads and blast loads. (There are deliberate but minor differences in some of the details, between Appendices B1 and B2, which are attributable to the general nature of B1 versus the more specific nature of B2 (home basement shelter for fallout and only low blast levels.)

c. For checking adequacy of designed columns in terms of their end bearing, and also for some comments on sill or bottom plate or blocking "design," see section "C. End Bearing and Sill/Bottom Plate Design," below.

B. <u>Numerical Example - Design Using Figure B1-2</u>

As an illustrative example: assume 2x4s are to be checked for their blast capacity in use as upgrading columns; Idaho White Pine is available, for which F_c is 650 psi and E is 1.2 million psi in Standard grade [1,Supplement,p.11; or table on p.B2-16, Appendix B2]; column height ℓ (L is also used in this section) is 96 in. Using table facing Figure B1-2A, L / d values are 64 for narrow (weaker) direction (d is 1.5 in.) and 27.4 for wide (stronger) direction (d is 3.5 in.); the larger L / d of 64 controls (Eq. 7). According to Eq. 8, this column is unusable (buckling hazard is too great).

The 2x4s can be used if supported at mid-height, in the weak direction, which is one alternative; however, let's take the other, that is, use a larger column by assuming that 4x4s are to be checked instead (normally, such a great increase would not be called for unless the designer is way off in his first guess, but poetic licence is taken here in order to demonstrate the two things that can happen in using Figure B1-2). Values for F_c and E are unchanged; L / d is 27.4 as before. Entering B1-2A with F_c of 650 and E of 1.2 million, read K as 29; since L / d is less than K, continue horizontally, interpolating horizontally between curves for L / d of 28 and 26, that is about 3/4 of the way from the 26 to the 28 curve (1.4 / 2 ≈ 3/4); go vertically from this interpolated point (about on 0.73 of bottom scale) to F_c of 650 in graph 3; then horizontally (along about the F_c' of 470 line) to the specific line for 4x4 (3.5x3.5) in graph 4 (no interpolating in this area!); then vertically to read P as 5.8 kips (5,800 pounds) and P_d as about 7.5 kips (7,500 pounds). (From calculations on page B2-28, Appendix B2: L / d is 27.43; K is 28.83; F_c' is 472, thus F_c'/F_c for bottom scale is 0.726; P is 5,782, from which P_d is 7,517.)[11]

[11] Numerical results are shown to illustrate the work, not to imply a degree of accuracy in design/analysis.

Now, assuming that the wood and grade are Douglas Fir-Larch and Construction, find F_c of 1,150 psi and E of 1.5 million psi from same sources as before. Using Figure B1-2A, graph 1 (interpolate vertically between E curves), find K as 24, meaning that L / d of 27.4 is greater than K, and Figure B1-2B must be used (instead of graphs 2 and 3 of B1-2A). Using B1-2B, enter with L / d of 27.4 and E of 1.5 million, read F_c' as 595; using graph 4 of B1-2A, enter with F_c' of 595 and read (for 4x4s) P of 7.3 kips (7,300 pounds) and P_d of 9.5 kips (9,500 pounds). (Again, from calculations on page B2-28, Appendix B2: L / d is 27.43 as before; K is 24.23; F_c' is 598; P is 7,327, from which P_d is 9,525.)

C. End Bearing and Sill/Bottom Plate Design

a. Adequacy of columns for end grain in bearing (top and bottom) should be checked. The method can be shown by example, using the two columns in the numerical example of the preceding section; Table B1-4 (fourth column) shows the design value for end bearing F_g for 4x4s as 1,390 psi for the Idaho White Pine and as 2,020 psi for the Douglas Fir-Larch. With the 4x4 (3.5x3.5) end area of 12.25 in.^{2}, the two example columns have end bearing capacities of 17,028 and 24,745 pounds if bearing is through metal plates, and 12,771 and 18,559 pounds if on less rigid materials (wood, concrete, etc.); see the comment along the bottom of Table B1-4. With the two example columns blast capacities P_d of 7,500 and 9,500 pounds, there is ample end bearing capacity even under the 75% limitation. Further, the design values F_g of Table B1-4 are subject to duration of loading factors [1,Sec.2.2.5.3]: increase the F_g values by 15% for loads of 2 months or less (as with fallout-shielding soil), and 100% for impact loads (such as air blast)! Thus, column adequacy for end grain in bearing might be concluded to be generally of no concern; nonetheless the checks should be made.

b. Sill or bottom plate "design" could be complex, involving as it does a beam with concentrated loads (the columns, plus foot plates if any) and supported by an elastic/plastic/fracturing foundation (the usual light, concrete floor slab). Such design is considered unwarranted for the purposes herein. It should be accepted that the floor slab will crack/break up and will be pushed down, under and adjacent to the bottom plate, should the upgraded floor system over the basement receive an air blast overpressure loading; such action absorbs energy. Recent tests indicate a very high floor slab resistance to column punching from blast duration loads (see page 18 of main text).

For the added loading of soil placed on the first floor for fallout shielding upgrading, it is doubted that the concrete basement floor will experience anything more than localized cracking, if that, under the following approach:

Table B1-4

END GRAIN IN BEARING (psi)

Design values for end grain in bearing parallel to grain on a rigid surface F_g in pounds per square inch

Species	Wet service conditions[1]	Dry service conditions[1]: Sawn lumber[2]: More than 4" thick	Dry service conditions[1]: Sawn lumber[2]: Not more than 4" thick[3]	Dry service conditions[1]: Glued laminated timber
[illegible]	1370	1510	2060	2400
Aspen	4[illegible]	820	1110	1300
[illegible]	890	980	1330	1560
[illegible]	1190	1310	1780	2080
Birch, Sweet and Yellow	1150	1260	1720	2010
[illegible]	620	690	930	1090
California Redwood (close grain)	1560	1720	2270	2620
California Redwood (open grain)	1150	1270	1670	1940
[illegible] Sitka Spruce	950	1040	1420	1660
[illegible] Species	950	1040	1420	1660
Cottonwood, Eastern	765	840	1150	1340
Douglas Fir-Larch (Dense)[4]	1570	1730	2360	2750
Douglas Fir-Larch[4]	1340	1480	2020	2350
Douglas Fir South	1220	1340	1820	2130
Eastern Hemlock-Tamarack[4]	1150	1270	1730	2020
Eastern Spruce	970	1070	1460	1700
Eastern White Pine[4]	900	1000	1360	1580
Eastern Woods	820	900	1230	1440
Engelmann Spruce-Alpine Fir	810	890	1220	1420
[illegible][4]	1110	1220	1670	1940
Hickory and Pecan	1370	1510	2050	2400
Idaho White Pine	930	1020	1390	1630
Lodgepole Pine	970	1060	1450	1690
Maple, Black and Sugar	1140	1260	1710	2000
[illegible]	1070	[illegible]	1600	1870
[illegible] Aspen	740	810	1110	1290
[illegible]	1040	1150	1570	1830
[illegible] Species	880	970	1320	1540
[illegible] White Cedar	740	810	1110	1290
[illegible]	1060	1160	1590	1850
[illegible]	[illegible]	1000	1370	1600
[illegible]	880	970	1320	1540
[illegible]	[illegible]	1090	1480	1730
[illegible]	[illegible]	1460	1990	2320
[illegible]	[illegible]	1690	2310	2690
[illegible]	[illegible]	1450	1970	2300
[illegible]	[illegible]	1040	1410	1650
[illegible]	[illegible]	1120	1530	1780
[illegible][3]	[illegible]	1140	1520	1750
[illegible]	[illegible]	1360	1860	2170
[illegible]	[illegible]	1030	1400	1630
[illegible]	[illegible]	890	1220	1420
[illegible]	[illegible]	890	1220	1420
[illegible]	890	980	1340	1560

[illegible] 4" × 4" [illegible] sawn lumber and 5" × 5" [illegible] glued laminated timber.

[illegible]

[illegible]

[illegible]

Source: National Design Specification for Wood Construction, 1977 Edition, National Forest Products Association, 1619 Massachusetts Avenue, N. W., Washington, D. C. 20036; art. 2.3, Table 2A.

Author comments: When stress in end-grain bearing exceeds 75% of above values, bearing shall be on a metal plate or strap, or on other durable, rigid, homogeneous material of adequate strength, per Source. (This criterion should be applied to both top and bottom ends of each stud or post.)

Use a bottom plate consisting of stress-graded selected dimension lumber (better than, say, utility grade), used flatwise, "2x" and at least as wide as the greater dimension of the columns (preferably wider) for a column spacing of 12 in. to 24 in.

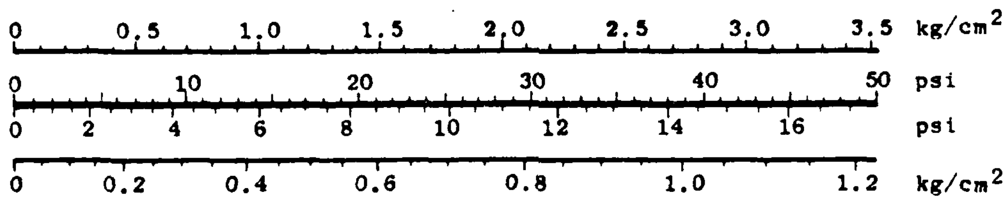
For greater column spacings, use any blocking available under each column flatwise and perhaps stacked, preferably with the bottom block wider than just prescribed for a single member bottom plate.

For column spacings of, say, 4 ft or more, one should leave off thinking of a continuous bottom plate as the column spacing grows larger and take recourse in use of a grillage under each column; the basic premise is to spread the load on the floor slab and underlying soil.

Peak Air Blast Resistance Capacity - Side-on versus Head-on

Peak applied air blast pressure resistance capacity (psi), determined above for the particular applied situation (with its design stresses from the grading association), was just that, as felt by the structure member. If the member is used so that the blast wave strikes it head-on, e.g., as if the member is part of the front wall of a building struck by the blast wave, then the blast wave is fully reflected, making it equivalent in loading force to a much stronger wave applied only side-on. To relate these two situations by putting both in terms of air blast peak free-field overpressure resistance (that is, out in the open, unaffected by structures), use the scales below:

FREE-FIELD OVERPRESSURE (WHEN APPLIED SIDE-ON)



FREE-FIELD OVERPRESSURE (WHEN APPLIED HEAD-ON OR FULLY REFLECTED)

For example, a free-field overpressure of 45 psi hitting the member side-on gives the same peak loading to the member as a free-field overpressure of 16 psi hitting the member head-on/fully reflected.

NOTATION

A = are of beam cross-section (in.^{2})

b = width of beam (in.)

C_d = drag coefficient (= ratio of drag pressure on object to dynamic/wind pressure in free field)

c,c′ = dimensionless coefficients

d = depth of wood beam (in.)

d = for columns, see figure in Table B1-3 (in.)

F_b = extreme fiber stress in bending (psi)

F_{db} = dynamic F_b (psi)

F_c = compression stress parallel to grain (psi)

F_c' = F_c corrected for buckling hazard (psi)

$F_{c\perp}$ = compression stress perpendicular to grain, or bearing stress (psi)

$F_{dc\perp}$ = dynamic $F_{c\perp}$ (psi)

F_v = horizontal shear stress (in wood) (psi)

F_{dv} = dynamic F_v (psi)

K = column slenderness ratio; see Table B1-3

L = span length of beam (center-center of supports unless otherwise indicated) (in.)

L = column overall length or unsupported length (see Table B1-3) (in.)

L′ = bearing length at each end of wood beam (in.)

M = bending moment (in.-lb)

P = total load (TL) column load capacity (normal use) (lb)

P_d = total load (TL) column load blast capacity (lb)

p_{dm} = peak (unit) value of applied (air blast) loading (psi)

p_r = peak reflected (air blast) overpressure (psi)

NOTATION (concluded)

p_{so} = peak side-on (air blast) overpressure (psi)

q = resistance of member, ultimate (psi)

q_b = bending q (psi)

q_n = resistance under normal use design (psi)

q_v = horizontal shear q (psi)

S = section modulus (in.^{3})

U = shock velocity (air blast) (fps)

V = vertical shear (lb)

w = load per unit length of beam (psf)

μ = ductility ratio (of maximum deflection to yield deflection)

REFERENCES

1. National Design Specification for Wood Construction, 1977 Edition, National Forest Products Association, 1619 Massachusetts Avenue, N.W., Washington, D.C., 20036, with 1980 supplement, Design Values for Wood Construction.

2. Timber Construction Manual, 2nd Edition, American Institute of Timber Construction, Englewood, Colorado, 1974 (or later edition).

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5. Newmark, N. M., Design of Openings for Buried Shelters, Report 2-67, U. S. Army Engineer Waterways Experiment Station, Vicksburg, Miss., July 1963.

6. Discussions with project consultant, Professor W. J. Hall, University of Illinois.

7. Newmark, N. M., "A Method of Computation for Structural Dynamics," Journal of the Engineering Mechanics Division, American Society of Civil Engineers, Proc. Paper 2094, July 1959; also ASCE Transactions, Vol. 127, Part I, Paper No. 3384, 1962.

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[12] Now SRI International.

[13] Now Federal Emergency Management Agency

Appendix B2

HOME BASEMENTS UPGRADING IN HOST AREAS

CONTENTS

TABLES

FIGURE

Appendix B2

HOME BASEMENTS UPGRADING IN HOST AREAS[1]

Introduction

With the advent of CRP (Crisis Relocation Planning) there is considerable interest in upgrading the floor over host area basements, in order that mass may be added to that floor (commonly soil over a sheet of plastic film) for fallout protection, perhaps even upgraded enough for say, 2 or 3 psi peak air blast overpressure protection, in addition to the added soil for fallout protection.

Commonly such floors are of wood subflooring (and often with wood finish flooring); carried by wood joists (with cross-bridging or blocking); in turn carried on the outer basement walls and a steel beam on a pipe column(s) running along the long-direction centerline of the basement. The pipe column(s) are supported on some kind of footing to spread the load, or infrequently on piling.

The prescribed floor live loading for a residential dwelling first floor is, and has been, 40 psf (pounds per square foot) [1(p.5-222)][2] under usual allowable design stresses [1,2].

All wood members that are depended upon (designed) to carry specified or calculated loads must be stress-graded by species and type of member, which data can in turn be used to look up allowable design stresses [2].

The discussion herein is for a "typical" basement, that is, one containing as many commonly-used features as possible; variations are many!

To discuss each of the structural elements just mentioned:

(1) The flooring must be not only strong enough for the 40 psf design live load plus the dead load of the flooring, both uniformly distributed loads, but also strong enough to carry safely (to the joists) the intermittent concentrated loads, such as that from heavy household

[1] Support for preparation of this appendix was provided under FEMA contracts with the Center for Planning and Research, Inc., Palo Alto, California and SRI International, Menlo Park, California.

[2] Brackets are used herein to indicate sources in the References list at the end of this appendix.

appliances. The result is that the flooring, considered by itself, is stronger than enough to handle the uniformly distributed dead load and 40 psf design live load.

(2) The floor joists - say, 2x6 or deeper,[3] and blocked or cross-bridged to prevent twisting along their bottom edges - are supported on an outer basement wall (along the long side of the basement, so that joist spans are in the shorter dimension of a rectangular house or of a rectangular portion of a larger house). Each floor joist is supported also on the steel beam discussed further below, and seldom if ever does one continuous wood joist run from one outer basement wall to the other but most often consists of two joists nailed together (for erection purposes, not flexural continuity) over the steel beam. Floor joists are used at intervals of 12 in. or 16 in. (rarely at 24 in.), measured center-to-center. Joist design must also consider (as prescribed by local building code) concentrated live loads, but the uniformly distributed 40 psf live load (also prescribed) usually controls design because of the ability of the joist blocking or cross-bridging to distribute concentrated and other loads to adjoining joists. In fact, the joists (and the flooring materials directly supporting loads) may be totally prescribed by the local building code, thus requiring no design work except perhaps for prudent checking.

(3) The supporting steel beam[4] for the floor joists, and its pipe[5] column(s) and footing(s):

(a) May carry only its share of the first floor live (40 psf) and dead (weight of materials built into the building) loads;

(b) More likely, however, the steel beam will carry directly above it an interior bearing partition that in turn carries half of the total ceiling load and attic load (latter, if habitable, 30 psf live load; uninhabitable, 20 psf [1]); and,

(c) Half the roof dead and live loads (latter is 20 to 100 or more psf, per local building code) are sometimes carried by an attic bearing partition (or row of columns) through the first floor bearing partition to the supporting steel beam under the first floor.

---

[3] Nominal dimensions; actual (finished) dimensions are smaller, i.e., a 2x6 is $1\frac{1}{2}$ in. x $5\frac{1}{2}$ in. (current) or 1-5/8 in. x $5\frac{1}{2}$ in. (formerly).

[4] What is described herein as a supporting steel beam may be, instead, a built-up or solid wood beam, but such substitution has little effect on the discussion in and purpose of this report.

[5] The most common support; however, another steel shape or a wood member(s) may be used.

Returning to the general discussion: A floor system, especially one with wood flooring and joists, is designed to meet both deflection and strength limits, with deflection often the controlling criterion (one dislikes walking on a springy floor, regardless of repeated assurances that it is safe). What this means, however, is that the floor is actually stronger than it needs to be TO CARRY LOADS, that is, to carry more than the prescribed design live and dead loads and still remain within safe design stresses. Further, the allowable stresses have built-in factors of safety that are reduced for unusual loads such as those from wind and earthquake, so can certainly be exploited for such an emergency need as supplementing the building's fallout and low level air blast protection. Finally, actual live loads expected to occur are usually less than prescribed (code) live loads described earlier; for example, a residential basement converted to a fallout shelter by adding soil a foot or so deep on the first floor is not apt also to have its usual live loads (people and furniture) to consider, certainly not to normal use levels.

Design/Analysis Annex Work

The Annex shows an analysis of an assumed floor joist system (2x8s on 16 in. centers) in its usual and an upgraded situation (support added at mid-span). Other supporting work included the design of top beams to transfer loads from the joists to columns spaced to carry $1\frac{1}{2}$ joists per column, and the design/analysis of several potential column sizes. The work and results are described in more detail for each member type in the following paragraphs.

(1) Annex sheets provide:

(a) Notation used.

(b) A tabulation of design stresses for one lower and one higher strength, widely available, wood species of two grades each, the latter to cover Dimension Lumber of two sizes: 2 in. to 4 in. thick, 4 in. wide; and 2 in. to 4 in. thick, 5 in. and wider.

(c) A set of wood beam design formulas for uniformly distributed loads: flexure; horizontal shear; end bearing length required; and, deflection limits as prescribed by building codes.

(d) A set of column design formulas.

(2) Another sheet (Usual Floor Joist Design) uses the above design formulas and allowable stresses to develop the allowable clear span for 2x8s on 16-in. centers, when using the lower and higher strength woods described above. Allowable spans (center-center of supports) proved to be 11 ft and 13 ft 4 in. for lower and higher strengths, respectively. The work uses a live load of 40 psf and a dead load of 10 psf.

But concern here is for host area shelter (see lower portion of the Annex sheet):

First, the deflection criteria may be ignored without hazard to the structure, which in this example increases the allowable live load for the higher strength wood (keeping the span unchanged);

Second, fallout shielding soil loads might be assumed as applied for two months or less, for which the design code allows an increase of 15 percent in live load;

Third, air blast loads only are impact live loads, for which the code allows a 100 percent increase (not additive to the above 15 percent!) in live load, to which one might add a 30 percent increase representing the current estimated factor of safety in wood grading (at 5 percent risk probability); and,

Fourth, combined shielding soil and air blast live loads may be handled as shown on the Annex sheet.

The approach leaves the residential basement undamaged by fallout protection, but risks damage if an attack occurs. The last admonition on the sheet, "re-check adequacy of end bearings," may be met by using Equations 10-12 on the Annex sheet, Wood Beam Design Formulas.

Dynamic factors for air blast are ignored in this appendix, because it is felt that a home getting less than, say, 5 psi air blast peak free field loading will feel an interior air blast pulse with a rise time long enough to make the effect no worse than that from a softly applied static load of short duration.

(3) The next Annex sheet begins the design work by assuming that these 2x8 joists on 16-in. centers have added mid-span supports installed for upgrading; thus the same lower and higher strength joists have their spans changed to 5 ft 6 in. and 6 ft 8 in. (both center to center), respectively. Deflection criteria are NOT used for the emergency purposes contemplated by upgrading work.

The sheet shows the design calculations and results: fallout soil (assuming 100 pcf) shielding capacity alone of soil 26.4 in. thick and 29.4 in. thick, for lower and higher strength woods, respectively; and, an example combined protection capacity of 12 in. deep soil (assuming 95 pcf), plus 1.96 psi and 2.36 psi blast overpressure (long duration), with the technique shown so that other combinations of fallout and blast protection may be calculated. Required bearing lengths are also shown on the sheet.

The adequacy of the flooring materials, to support these new total loads, was assumed to be sufficient for the reason stated in paragraph 1 of the Introduction. Such adequacy should be checked, of course; the approach for the flooring materials is the same as for any wood beam examined with the use of the Annex sheet on beam formulas.

(4) The work included the design of top beams (3 sheets), showing those with capacities of interest for joist loads of the range encountered above. Results[6] of the top beam designs are shown in Table B2-1. Top beam capacities shown exclude the allowable stress increases (15 and 100 percent) for short duration loads discussed above under floor joist design; similar increases apply to top beams.

(5) The work also covered examination of various sizes of columns (3 sheets) for their load capacity, with the rectangular, non-square sizes also checked for capacity where mid-height supports (and at third-points in some cases) in the weak direction have been added. Results are shown in Table B2-2.

Figure B2-1 shows schemes for floor joists upgrading: one with wood posts at a spacing of $1\frac{1}{2}$ floor joists per post, with top beams to transfer joist loads to the posts; the other with a stud directly under each joist and using a top (or tie) plate to prevent twisting of the bottom edges of the joists. The top (or tie) plate may be unneeded if the joists have adequate blocking or cross-bridging (and each joist bottom edge has enough bearing area for the top of a stud).

Applications

Obviously, if one could go into a house and identify fully the wood floor joist material used (species, size and grade of wood) and happened to have a copy of the applicable grading rules (shirt-pocket size for the particular grading association OR Reference [2] with back editions), design stresses could be applied as they have been in the preceding section and the Annex, leading to analysis of the joists and design of joist upgrading and of top beams and columns. Such a situation is unlikely to apply.

An alternate approach might be as described in the following paragraphs:

(1) Assume that the floor over the basement was built according to a building code calling for 40 psf live load on the first floor and allowing 10 psf for dead load (weight of flooring materials and joists). (Many buildings were not built according to code or the code was not enforced, but many are stronger (especially if owner built) and probably a lot are weaker (e.g., as in speculative housing built to cut corners).)

[6] Numerical results are shown to illustrate the work, not to imply a degree of accuracy in design/analysis.

Table B2-[illegible]

TOP BEAM CAPACITY

	Joist Spacing	[illegible] Lower	[illegible] Upper	
Top Beam Capacities ([illegible] joists per [illegible])				
Triple 2x8s, edgewise	12"			
	16"	[illegible]	[illegible]	[illegible]
	24"			
Quadruple 2x6s, edgewise	12"			
	16"	2588	5624	[illegible] joist
	24"			

NOTE: See Annex sheets (3), Top Beams, for design method including required bearing lengths on top and bottom surfaces of these two top beam schemes.

Table B2-2

COLUMN CAPACITIES

		Member Strengths	
		Lower	Higher
	Column Capacities (8 ft long) in pounds:		
	2x4 column (too long and slim, per code)		
a.	2x4 column, with mid-ht support: (meaning laterally, in weak direction)	1848	2305
b.	4x4 column:	5782	7327
	2x6 column (too long and slim, per code)		
c.	2x6 column, with mid-ht support:	3143	4348
d.	2x6 column, with 3rd-point supports:	5346	8621
	2x8 column (too long and slim, per code)		
e.	2x8 column, with mid-ht support:	4143	5731
f.	2x8 column, with 3rd-point supports:	7047	11364
g.	4x6 column:	9914	13822
h.	4x6 column, with mid-ht support:	13302	22292

NOTE: Above capacities should be reduced for the column's own weight and may be increased for various live load durations (see Annex sheet 2 of Columns for Upgrading).

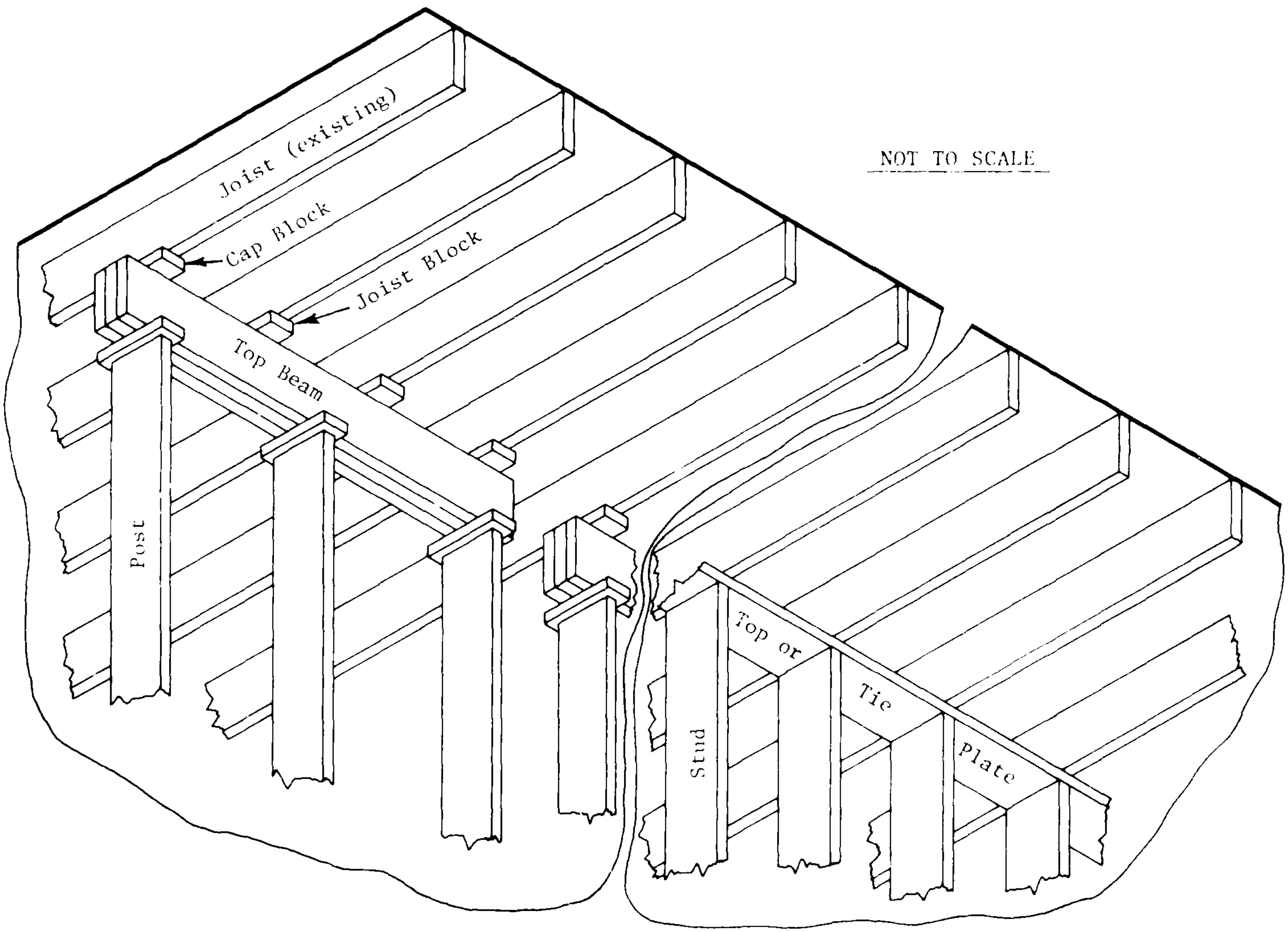

Figure B2-1A FLOOR JOISTS UPGRADING

NOT TO SCALE

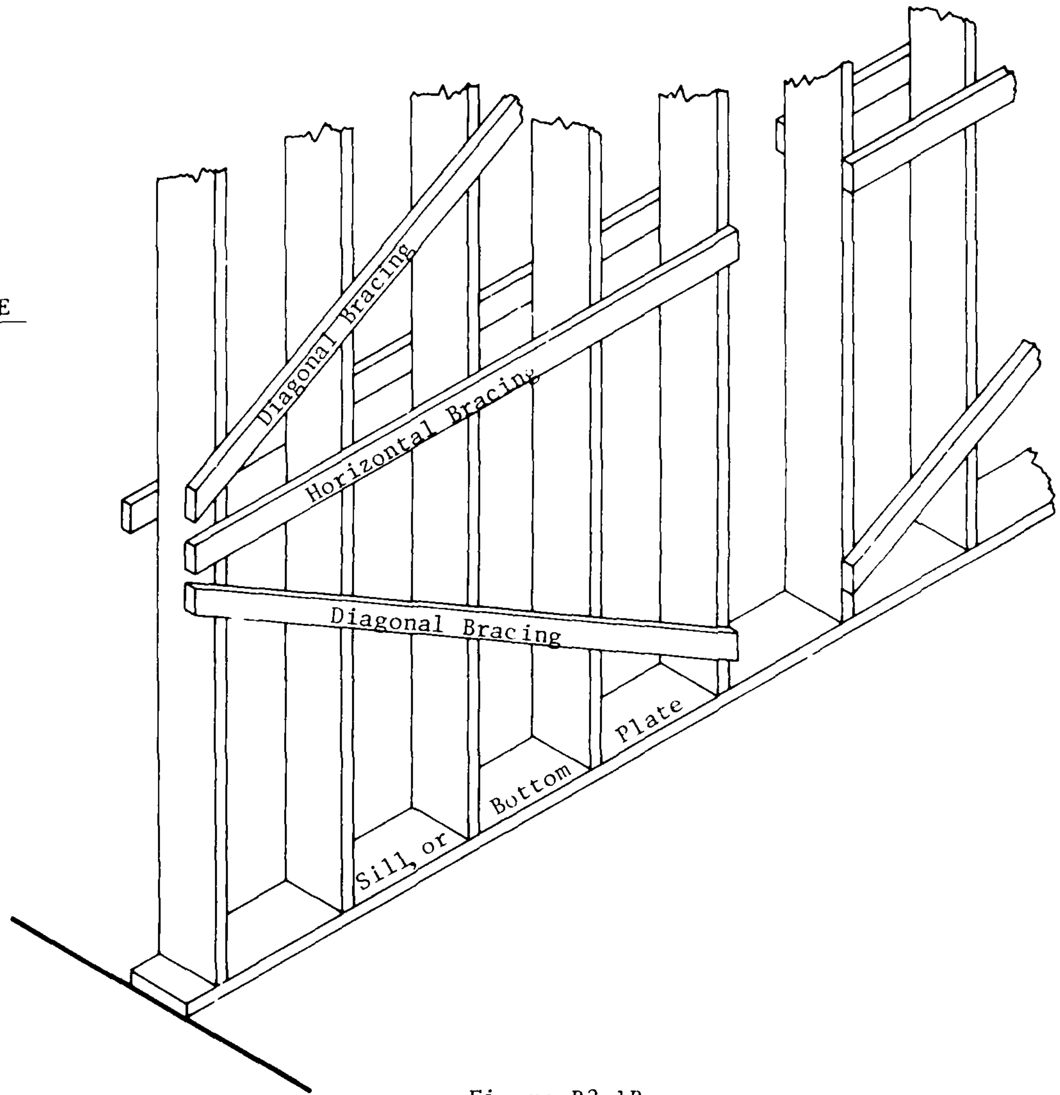

Figure B2-1B

(2) Measure the actual joist width b, depth d, and center-to-center span[7] L of the joist being checked. Also measure the c-c. spacing j between joists. Use all four dimensions in inches, unless otherwise indicated below.

(3) Enter Table B2-3, using the nominal b and d (see Table B2-4), and actual j and L dimensions (all in inches, except L in feet and inches here); read the values for E and F_b. (For example: 2x8s on 16 in. centers, span 11 ft 8 in., read E as 1,200,000 psi and F_b as 1,040 psi.) Interpolate as necessary if the table doesn't include the specific field measurements obtained. If the actual b and d measurements agree with the column of Table B2-4 headed "Actual (older)," multiply the E value read in Table B2-3 by the "Corr. Factor" shown in Table B2-4.

As an alternative to the Table B2-3 approach, use the following formulas (no "Corr. Factor"; all actual dimensions, in inches):

$$E = 15.625\ (j / b)\ (L / d)^3 \quad \text{(psi)}$$

$$F_b = 0.26042\ (j / b)\ (L / d)^2 \quad \text{(psi)}$$

Both table and formula approaches assume floor designs based on 40 psf live load, 10 psf dead load, and live load deflection limited to 1/360th of the span.

(4) Values for F_v and $F_{c\perp}$ (both in psi) can be calculated from the following:[8]

$$F_v = 22.6736 + 71.3296\ (E / 1{,}000{,}000)$$

$$F_{c\perp} = -135.76 + 313.498\ (E / 1{,}000{,}000)$$

---

[7] Joist span is measured between the centers of its supports, usually a steel beam down the basement center and an outer basement wall.

[8] Least squares regression equations shown were obtained by use of elasticity moduli and allowable stress values [2] for six grades each of eight species of widely available construction woods: Douglas Fir-Larch; Western Hemlock; Ponderosa, Sugar, Idaho White, and Lodgepole Pines; Southern Pine; and Northern Pine; all when used at 19 percent or less moisture content. (Number of significant digits shown does not imply a degree of accuracy in the results.) Range of E values used was 1.0 to 1.8 million psi; of F_v, 100-140 psi; of $F_{c\perp}$, 190-405 psi. With 36 pairs of values used for each of the two equations, the range of percentage differences between each actual value and value calculated with the applicable equation was: for F_v, 16.9 percent to -18.3 percent; for $F_{c\perp}$, 33.9 percent to -37.3 percent.

Table B2-3

ALLOWABLE SPANS FOR FLOOR JOISTS
(40 psf LL and 10 psf DL)

DESIGN CRITERIA: Deflection - For 40 lbs. per sq. ft. live load. Limited to span in inches divided by 360. Strength - Live load of 40 lbs. per sq. ft. plus dead load of 10 lbs. per sq. ft. determines the required fiber stress value.

JOIST SIZE (IN)	JOIST SPACING (IN)	Modulus of Elasticity, "E", in 1,000,000 psi													
		0.8	0.9	1.0	1.1	1.2	1.3	1.4	1.5	1.6	1.7	1.8	1.9	2.0	2.2
2x6	12.0	8-6 720	8-10 780	9-2 830	9-6 890	9-9 940	10-0 990	10-3 1040	10-6 1090	10-9 1140	10-11 1190	11-2 1230	11-4 1280	11-7 1320	11-11 1410
	16.0	7-9 790	8-0 860	8-4 920	8-7 980	8-10 1040	9-1 1090	9-4 1150	9-6 1200	9-9 1250	9-11 1310	10-2 1360	10-4 1410	10-6 1460	10-10 1550
	24.0	6-9 900	7-0 980	7-3 1050	7-6 1120	7-9 1190	7-11 1250	8-2 1310	8-4 1380	8-6 1440	8-8 1500	8-10 1550	9-0 1610	9-2 1670	9-6 1780
2x8	12.0	11-3 720	11-8 780	12-1 830	12-6 890	12-10 940	13-2 990	13-6 1040	13-10 1090	14-2 1140	14-5 1190	14-8 1230	15-0 1280	15-3 1320	15-9 1410
	16.0	10-2 790	10-7 850	11-0 920	11-4 980	11-8 1040	12-0 1090	12-3 1150	12-7 1200	12-10 1250	13-1 1310	13-4 1360	13-7 1410	13-10 1460	14-3 1550
	24.0	8-11 900	9-3 980	9-7 1050	9-11 1120	10-2 1190	10-6 1250	10-9 1310	11-0 1380	11-3 1440	11-5 1500	11-8 1550	11-11 1610	12-1 1670	12-6 1780
2x10	12.0	14-4 720	14-11 780	15-5 830	15-11 890	16-5 940	16-10 990	17-3 1040	17-8 1090	18-0 1140	18-5 1190	18-9 1230	19-1 1280	19-5 1320	20-1 1410
	16.0	13-0 790	13-6 850	14-0 920	14-6 980	14-11 1040	15-3 1090	15-8 1150	16-0 1200	16-5 1250	16-9 1310	17-0 1360	17-4 1410	17-8 1460	18-3 1550
	24.0	11-4 900	11-10 980	12-3 1050	12-8 1120	13-0 1190	13-4 1250	13-8 1310	14-0 1380	14-4 1440	14-7 1500	14-11 1550	15-2 1610	15-5 1670	15-11 1780
2x12	12.0	17-5 720	18-1 780	18-9 830	19-4 890	19-11 940	20-6 990	21-0 1040	21-6 1090	21-11 1140	22-5 1190	22-10 1230	23-3 1280	23-7 1320	24-5 1410
	16.0	15-10 790	16-5 860	17-0 920	17-7 980	18-1 1040	18-7 1090	19-1 1150	19-6 1200	19-11 1250	20-4 1310	20-9 1360	21-1 1410	21-6 1460	22-2 1550
	24.0	13-10 900	14-4 980	14-11 1050	15-4 1120	15-10 1190	16-3 1250	16-8 1310	17-0 1380	17-5 1440	17-9 1500	18-1 1550	18-5 1610	18-9 1670	19-4 1780

[1]The required extreme fiber stress in bending, F_b, in pounds per square inch is shown below each span.

[2]Use single or repetitive member bending stress values (F_b) and modules of elasticity values (E).

[3]For more comprehensive tables covering a broader range of bending stress values (F_b) and Modulus of Elasticity values (E), other spacing of members and other conditions of loading, see the Uniform Building Code.

[4]The spans in these tables are intended for use in covered structures or where moisture content in use does not exceed 19 percent.

Source: Dwelling Construction Under the Uniform Building Code, 1976 Edition, International Conference of Building Officials, Whittier, California 90601

Table B2-4

JOIST DIMENSIONS - WIDTH x DEPTH (in.)

Nominal	Actual (current)	Actual (older) & E Corr. Factor	
2x6	1.5x5.5	1-5/8 x 5-1/2	0.923
2x8	1.5x7.25	1-5/8 x 7-1/2	0.834
2x10	1.5x9.25	1-5/8 x 9-1/2	0.852
2x12	1.5x11.25	1-5/8 x 11-1/2	0.864

(5) Plan to install an additional support line under the mid-span points of all joists. Identify the measurements made under paragraph 2 as b for actual width of joist, d for actual depth, j for joist spacing center-to-center (c-c.), and change L to HALF the c-c. span measured for the full, original joist (paragraph 2), all in inches. Calculate the allowable unit total load TL on the joists with new mid-span supports as follows (TL in pounds per square foot (psf)):

$$TL = 154\ bdF_v\ /\ (j\ (L-2d))$$

$$\text{or} = 192\ bd^2F_b\ /\ (jL^2)$$

whichever is less.

About 10 psf of this TL will roughly cover the flooring materials, joists, top beam if any, and posts or studs, so that the dead load DL of 10 psf (approximately) can be subtracted from the TL to obtain the allowable live load LL on the floor. The equations exclude any allowed stress increases for load duration of two months or less (15 percent) or impact (100 percent), or for removing the design factor of safety (30 percent). Such increases are handled as described in paragraph 2 of the preceding section, Design/Analysis Annex Work; the comments there about the adequacy of the flooring to carry the above TL between floor joists, also apply here.

(6) If p is the total load (pounds) per joist, as transmitted to the top beam if any, or directly to a stud, it equals the TL (as calculated, but plus any allowed percentage increases, both as just described) times the contributory (floor) area, jL; that is:

$$p = (TL)\ jL\ /\ 144.$$

(7) The required joist bearing area, bL′ (actual width b; bearing length L′ along joist; both in inches) on the bottom of the joist at mid-span can be calculated from the following:

$$bL' = p\ /\ F_{c\perp}$$

with L′ in inches, using $F_{c\perp}$ (psi) as that for the joist (paragraphs 2-4 above).

If the bL′ required by the joist is not provided by the stud or top beam, a wood block may be needed under the joist (even if the block crushes under blast loading, it will be absorbing energy), or recourse to a steel plate may be necessary.

The required bearing area on the top and underside of a top beam can be similarly calculated, but using the appropriate $F_{c\perp}$ value for the top beam wood; calculation sheets in the Annex also show the methods.

It is assumed that upgrading materials used, presumably obtained from lumberyards, will be known as to species, size and grade of wood, and thus that allowable design values - F_b (both single- and repetitive-member use), F_c, $F_{c\perp}$, F_v and E - are obtainable from Reference [2] or the applicable association grading rules. Reference [2] includes associations' addresses, and it is urged that the rules be used, time permitting, to verify the grading of each piece of lumber.

(8) Top beams may be omitted by putting a stud under each joist, using a scab board on each side of the stud-joist junction if both members are the same width, or using a top (or tie) plate (Figure B2-1).

If top beams are to be used, posts should be located under the two end joists to be served by each top beam. If the post spacing is to be other than $1\frac{1}{2}$ times the joist spacing, a new top beam design should be prepared by a competent civil engineer with wood design experience. For post spacing at $1\frac{1}{2}$ times the joist spacing, design work has been done, as reported in the preceding section and in Table B2-1. The following calculations may be used to generalize, based on the column spacing at $1\frac{1}{2}$ times the joist spacing ONLY.[9]

Four top beam designs (2 sizes, each at two strengths) are shown in Table B2-1; they may be used as follows:

Top Beam Capacities ($1\frac{1}{2}$ floor joists per column):

Triple 2x8s, edgewise:[9]

$$p = 25.6\ F_v$$

Quadruple 2x6s, edgewise:[9]

$$p = 25.9\ F_v$$

with the load-per-joist p in pounds, excluding all add-on percentages related to load duration and factor of safety.

(9) Eight column designs (all 8 ft long and using "2x" or "4x" members) are identified as "a." through "h." in Table B2-2; some may be generalized as follows (P is allowable axial load in pounds):

a. $P = 0.00154\ E$ e. $P = 0.00319\ E$

c. $P = 0.00242\ E$

b., d., f., g., and h.: Design column, using "Column Formulas" of Annex.

[9] Checked for joist spacings of 24 in. or less.

Design values for E and F_c should be obtained as described in paragraph 7 above.

The column capacities in Table B2-2, including those from formulas just above, include no duration of load increases - see NOTE on Table B2-2.

Adequacy of these columns for end grain in bearing (Idaho White Pine = 1390 psi and Douglas Fir-Larch = 2020 psi applied to the lower and higher member strengths, respectively, see Table B2-5 fourth column) has been checked and all are adequate. Note that the F_g design values of Table B2-5 are subject to the same duration of loading increases (15% and 100% for example), as are the column capacities of Table B2-2.

(10) Sill or bottom plate "design" could be complex, involving a beam with concentrated loads (the columns, plus foot plates if any) and supported by an elastic/plastic/fracturing foundation (the usual light, concrete floor slab). Such design is considered unwarranted for the purposes herein. It should be accepted that the floor slab will crack/break up and will be pushed down, under and adjacent to the bottom plate, should the upgraded floor system over the basement receive an air blast overpressure loading; such action absorbs energy. Recent tests show a very high floor slab resistance to column punching from blast duration loads (see page 18 of main text).

For the added loading of soil placed on the first floor for fallout shielding upgrading, it is doubted that the concrete basement floor will experience anything more than localized cracking, if that, under the following approach:

Use a bottom plate consisting of stress-graded selected dimension lumber (better than, say, utility grade), used flatwise, "2x" and at least as wide as the greater dimension of the columns (preferably wider) for a column spacing of 12 in. to 24 in.

For greater column spacings, use any blocking available under each column flatwise and perhaps stacked, preferably with the bottom block wider than just prescribed for a single member bottom plate.

For column spacings of, say, 4 ft or more, one should leave off thinking of a continuous bottom plate as the column spacing grows larger and take recourse in use of a grillage under each column; the basic premise is to spread the load on the floor slab and underlying soil.

Design graphs for wood beams and columns are included in Appendix B1 herein.

Table B2-5

END GRAIN IN BEARING (psi)

Design values for end grain in bearing parallel to grain on a rigid surface F_g in pounds per square inch

Species	Wet service conditions[1]	Dry service conditions[1]		
		Sawn lumber[2]		Glued laminated timber
		More than 4" thick	Not more than 4" thick[3]	
Ash, Commercial White	1370	1510	2060	2400
Aspen	740	820	1110	1300
Balsam Fir	890	980	1330	1560
Beech	1190	1310	1780	2080
Birch, Sweet and Yellow	1150	1260	1720	2010
Black Cottonwood	620	690	930	1090
California Redwood (Close grain)	1560	1720	2270	2620
California Redwood (Open grain)	1150	1270	1670	1940
Coast Sitka Spruce	950	1040	1420	1660
Coast Species	950	1040	1420	1660
Cottonwood, Eastern	765	840	1150	1340
Douglas Fir-Larch (Dense)[4]	1570	1730	2360	2750
Douglas Fir-Larch[4]	1340	1480	2020	2350
Douglas Fir South	1220	1340	1820	2130
Eastern Hemlock-Tamarack[4]	1150	1270	1730	2020
Eastern Spruce	970	1070	1460	1700
Eastern White Pine[4]	900	1000	1360	1580
Eastern Woods	820	900	1230	1440
Engelmann Spruce-Alpine Fir	810	890	1220	1420
Hem-Fir[4]	1110	1220	1670	1940
Hickory and Pecan	1370	1510	2050	2400
Idaho White Pine	930	1020	1390	1630
Lodgepole Pine	970	1060	1450	1690
Maple, Black and Sugar	1140	1260	1710	2000
Mountain Hemlock	1070	1170	1600	1870
Northern Aspen	740	810	1110	1290
Northern Pine	1040	1150	1570	1830
Northern Species	880	970	1320	1540
Northern White Cedar	740	810	1110	1290
Oak, Red and White	1060	1160	1590	1850
Ponderosa Pine-Sugar Pine	910	1000	1370	1600
Red Pine	880	970	1320	1540
Sitka Spruce	990	1090	1480	1730
Southern Cypress	1330	1460	1990	2320
Southern Pine (Dense)	1540	1690	2310	2690
Southern Pine	1320	1450	1970	2300
Spruce-Pine-Fir	940	1040	1410	1650
Sweetgum and Tupelo	1020	1120	1530	1780
Western Cedars[4]	1040	1140	1520	1750
Western Hemlock	1240	1360	1860	2170
Western White Pine	930	1030	1400	1630
White Woods (Western Woods)	810	890	1220	1420
West Coast Woods (Mixed Species)	810	890	1220	1420
Yellow Poplar	890	980	1340	1560

1 Wet and dry service conditions are defined in 4.1.4 for sawn lumber and 5.1.5 for glued laminated timber.

2 Applies to sawn lumber members which are at a moisture content of 19 percent or less when full design load is applied, regardless of moisture content at time of manufacture.

3 When 4 inch or thinner sawn lumber is surfaced at a moisture content of 15 percent or less and is used under dry service conditions, the values listed for glued laminated timber may be applied.

4 Values also apply when species name includes the designation "North."

Source: National Design Specification for Wood Construction, 1977 Edition, National Forest Products Association, 1619 Massachusetts Avenue, N. W., Washington, D. C. 20036; art. 2.3, Table 2A.

Author Comments: When stress in end-grain bearing exceeds 75% of above values, bearing shall be on a metal plate or strap, or on other durable, rigid, homogeneous material of adequate strength, per Source. (This criterion should be applied to both top and bottom ends of each stud or post.)

ANNEX

Stress-graded (visually only) lumber species-sizes-grades [1] commonly available in local lumberyards were narrowed down to the following, categorized as "higher strength" and "lower strength" for use in an earlier study [4,p.C-3&5]:

Allowable Design Stresses in Normal Use (psi)

Size & Grade	F_b [2]	F_v [3]	$F_{c\perp}$ [4]	F_c	E	F_g [5]
2 in. to 4 in. th., 4 in. wide						
Construction [6]	1200 (1050)	140	385	1150	1500000	2020
Standard [7]	450 (400)	100	190	650	1200000	1390
2 in. to 4 in. th., 5 in. & wider [8]						
No. 1 [6]	1750 (1500)	140	385	1250	1800000	2020
No. 2 [7]	925 (850)	100	190	725	1300000	1390

---

[1] Reference [2] contains a full listing of wood species, sizes, grades, design values, etc. However, it lacks two needed tables from Reference [3], which are included herein as Tables B2-5 and B2-6.

All of above is Dimension Lumber; surfaced dry or surfaced green; used at 19 percent maximum moisture content.

[2] Repetitive-member use; single-member use values in parentheses [2,p.20]. Both uses are edgewise; for flatwise-use (increase) factors, see Reference [2,p.16].

[3] Values taken from Reference [3,art.3.4.4.2]; see Table B2-6.

[4] For bearing lengths L′ less than 6 in. long and not nearer than 3 in. to end of member, correct $F_{c\perp}$ by $(1 + 0.375 / L')$ [3,art.3.11.2.2].

[5] Values taken from Reference [3,Table 2A,p.7]; see Table B2-5.

[6] "Higher strength"; stresses for Douglas Fir-Larch [2].

[7] "Lower strength"; stresses for Idaho White Pine, but useful (practically) for other Western Pines (Ponderosa, Sugar and Lodgepole) [2].

[8] But not more than 12 in. [5,p.16-17].

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Table B2-6

ALLOWABLE HORIZONTAL SHEAR VALUES F_v (psi)

	Maximum Moisture Content		
	Unseasoned	19 percent	15 percent
Aspen	85	90	95
Balsam Fir	85	95	95
Black Cottonwood	70	75	80
California Redwood	115	120	130
Coast Sitka Spruce	90	95	100
Coast Species	90	95	100
Douglas Fir- Larch	130	140	145
Douglas Fir–South	130	140	145
Eastern Hemlock–Tamarack	120	130	135
Eastern Spruce	95	105	110
Eastern White Pine	90	95	100
Eastern Woods	85	95	95
Engelmann Spruce/Alpine Fir	95	105	110
Hem–Fir	105	110	115
Idaho White Pine	95	100	105
Lodgepole Pine	95	105	110
Mountain Hemlock	130	140	150
Northern Aspen	90	95	100
Northern Pine	100	105	110
Northern Species	90	95	100
Northern White Cedar	85	95	100
Ponderosa Pine–Sugar Pine	100	105	110
Red Pine	100	110	115
Sitka Spruce	105	115	120
Southern Pine	125	135	145
Spruce–Pine–Fir	95	105	110
Western Cedar	100	105	110
Western Hemlock	125	135	145
Western White Pine	90	100	105
White Woods (Western Woods, West Coast Woods, Mixed Species)	95	100	105

Source: National Design Specification for Wood Construction, 1977 Edition, National Forest Products Association, 1619 Massachusetts Avenue, N.W., Washington, D. C. 20036; art. 3.4.4.2

Author Comments: Use column of above table that specifies "19 percent" moisture content. For wood species not shown, consult "Design Values for Wood Construction," 1980 Supplement to above Source.

WOOD BEAM DESIGN FORMULAS - UNIFORMLY DISTRIBUTED LOADS

With simple end supports and with continuous supports, beam span L for horizontal shear design is measured center-to-center (c-c.) of supports. L is the same with continuous supports for flexure and deflection design; but with simple end supports, beam span L for flexure and deflection design should be the beam length between support faces plus half the REQUIRED bearing length L′ at each end [3,art.3.2.1,p.8]. For simplification, however, <u>L will be taken as c-c. of supports for all purposes herein.</u>

Constant C is 360 under LL (live loads) only, and is 240 under LL plus DL (live plus dead loads). Check both conditions.

Constant C′ varies with support conditions as shown: SS is simple supports; PC is propped contilever; and FF is fixed-fixed.

	C′ Constants		
	SS	PC	FF
<u>FLEXURE</u>:	8	8	12

$M = wL^2 / C' = F_bI / c = F_bbd^2 / 6$ $\quad (I=bd^3/12)$ $\quad (c=d/2)$ (1)

$L^2 = C'M / w = C'F_bbd^2 / (6w)$ (2)

$w = C'M / L^2 = C'F_bbd^2 / (6L^2)$ (3)

	SS	PC	FF
<u>HORIZONTAL SHEAR</u>:	2	1.6	2

$V = w\ (L - 2d) / C' = 2\ bdF_v / 3$ (4)

$L = (C'V / w) + 2d = (2C'bdF_v / (3w)) + 2d$ (5)

$w = C'V / (L - 2d) = 2C'bdF_v / (3\ (L - 2d))$ (6)

	SS	PC	FF
<u>DEFLECTION</u>:	5/384	1/185	1/384

$y = L / C = C'wL^4 / EI = 12C'wL^4 / (Ebd^3)$ (7)

$L^3 = Ebd^3 / (12C'Cw)$ (8)

$w = (d / L)^3\ Eb / (12C'C)$ (9)

BEAM BEARING LENGTH L′ (at each end):[9] 2 (10) 2

If $L' < 6$ in. AND bearing area is > 3 in. from end:

$$R = V = wL / C' = bF_{c\perp} (L' + 0.375) \qquad (10)$$

$$L' = (wL / (bC'F_{c\perp})) - 0.375 \qquad (11)$$

$$w = (bC'F_{c\perp} / L)(L' + 0.375) \qquad (12)$$

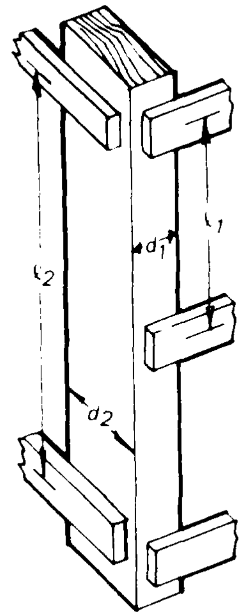
If $L' > 6$ in. OR bearing area is < 3 in. from end: Drop 0.375 from Eq. 10-12. (For simplification, consider dropping the 0.375 in all cases.)

---

[9] It is recommended that L′ be not less than 2 in.

[10] $C' = 1.6$ at fixed end; $C' = 2.67$ at simple support or "propped" end.

COLUMN FORMULAS - SIMPLE SOLID COLUMN DESIGN [3]
(Pin-ended conditions assumed)



ℓ_1 and ℓ_2 = Distances between points of lateral support of column in planes 1 and 2, inches

d_1 and d_2 Dimensions of column in planes of lateral support, inches

For conditions shown in the sketch:

Use <u>larger</u> of L_1 / d_1 and L_2 / d_2 as <u>the</u> L / d for design (13A)

Slenderness ratio L / d must be ≤ 50 (13B)

<u>For $L / d \leq 11$</u>: $F_c' = F_c$ (14)

<u>For $L / d > 11$, but $\leq K$</u>:

$K = 0.671 \sqrt{E / F_c}$ (15)

$F_c' = F_c (1 - 1/3 (L / dK)^4)$ (16)

<u>For $L / d \geq K$</u>: $F_c' = 0.3E / (L/d)^2$ (17)

$P = AF_c' = bdF_c'$ (18)

Usual Floor Joist Design

Try 2x8 (1.5" x 7.25") floor joist, on 16" centers. Use: floor live load, LL = 40 psf; floor dead load, DL = 10 psf.
Thus $DL + LL = (10+40) \times \frac{16''}{12''} \times \frac{1}{12''/ft} = \frac{50}{9}$ pli = w.

Find usable c-c spans, for "lower" and "higher" member strengths — see Annex 1st page (above).

	Lower L	Higher L	(Strength Wood)
Flexure: Eq. 2: $L^2 = 8F_b \times 1.5 \times \overline{7.25}^2 / (6 \times \frac{50}{9}) =$ $\begin{bmatrix} 925 \\ 1750 \end{bmatrix}$	132"	182"	
Horiz. Shear: Eq. 5: $L = 2 \times 2 \times 1.5 \times 7.25 \times F_v / (3 \times \frac{50}{9}) + 2 \times 7.25 =$ $\begin{bmatrix} 100 \\ 140 \end{bmatrix}$	276	380	
Deflection: Eq. 8: $L^3 = E \times 1.5 \times \overline{7.25}^3 / (12 \times \frac{5}{384} \times Cw) =$ $\begin{bmatrix} 1.3 \times 10^6 \\ 1.8 \times 10^6 \end{bmatrix}$ $\begin{bmatrix} 240 \\ 360 \end{bmatrix} \begin{bmatrix} 50/9 \\ 40/9 \end{bmatrix}$	153	170	C = 240
	144	160	C = 360
	L'	L'	
End Bearing: Eq. 11: $L' = \frac{50}{9} \times L / (1.5 \times 2 \times F_{cL})$ $\begin{bmatrix} 132 \\ 160 \end{bmatrix}$ $\begin{bmatrix} 190 \\ 385 \end{bmatrix}$	1.29 USE 2" (min.)	0.77 USE 2" (min.)	

Least L for lower, and least L for higher are 132" (flexure controlling) and 160" (deflection, with C = 360, controlling), respectively.
These 2 L values are used for End Bearing calc.'s.

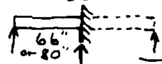
Note: (a) For upgrading design, ignore deflection as a design criterion.
(b) Above two L values agree exactly with Table B1-3.

HOST SHELTER USE:			
L (c-c supports) =	132	160	in.
But, ign. deflection, so (Eq. 3) $w = 8F_b \times 1.5 \times \overline{7.25}^2 / (6L^2)$ $\begin{bmatrix} 925 \\ 1750 \end{bmatrix}$ $\begin{bmatrix} 132 \\ 160 \end{bmatrix}$ =	50/9	7.19	pli TL
×144/16 =	50	65	psf TL
	−10	−10	" DL
	40	55	" LL
A. Fallout only (assuming soil @ 95 pcf); +15% for loads < 2 mos.: ×1.15 =	46	63	" "
or	5.8	8.0	in. th. soil
B. Combined: Fallout: Use ≤ soil LL in A.; say, 4.5 in. @ 100 pcf used:	38	38	psf LL (soil)
	46−38 = 8	63−38 = 25	psf LL (soil) (capacity remaining)
Blast capacity (remove 15% of A; instead use +100% for impact loads and +30% factor of safety given up) = $\frac{2.0 \times 1.3}{1.15} \times \begin{bmatrix} 8 \\ \text{or} \\ 25 \end{bmatrix} =$	18	57	psf LL (blast)
÷144 =	0.13	0.39	psi " "

Re-check end bearing adequacy.

Upgrading Floor Joist Design

Try same 2x8 (1.5 x 7.25) floor joist, on 16" centers, but with an added mid-span support — same 2 strengths — spans now (132÷2=) 66" and (160÷2=) 80" for "lower" and "higher" strengths, respectively.
Find w. Use DL = 10 psf, as before. PC supports.



	Lower w	Higher w	
Flexure: Eq. 3: $w = 8F_b\,1.5\times 7.25^2/(6L^2)$ =	22.3	28.7	pli TL
$\begin{bmatrix}925\\1750\end{bmatrix}$ $\begin{bmatrix}66\\80\end{bmatrix}$ (j=16) $\times\frac{144}{16}$, TL =	201	259	psf TL
DL =	−10	−10	" DL
LL =	191	249	" LL
Horiz. shear: Eq. 6: $w = 2\times 1.6\times 1.5\times 7.25\,F_v/(3\times(L-2\times 7.25))$ =	22.5	24.8	pli TL
$\begin{bmatrix}100\\140\end{bmatrix}$ $\begin{bmatrix}66\\80\end{bmatrix}$ TL =		223	psf TL
DL =		−10	" DL
LL =		213	" LL

UPGRADING:

	Lower	Higher	
A. Fallout shielding only (assume soil @100 pcf); add 15% for loads ≤2 mos. duration (Ref. 3, art. 2.2.5.3) LL =	191	213	psf LL
(x1.15) LL =	220	245	psf(soil) LL
Soil th. =	26.4	29.4	in. soil
[$w = (LL + DL)\times 16/144$ =	25.6	28.3	pli TL]
	191	213	psf LL
B. Combined: Fallout: Use ≤ soil LL in A. x1.15 =	220	245	psf(soil) LL
Assuming 12" th. 95 pcf soil; +15% as before:	−95	−95	" " "
	125	150	psf(soil) LL
		capacit. remaining	
Blast capacity (remove 15% of A.; instead use +100% for impact loads and +30% factor of safety given up) = $\frac{2.0\times 1.3}{1.15}\times\begin{bmatrix}125\\ \text{or}\\150\end{bmatrix}$ =	283	339	psf(blast) LL
÷144	1.96	2.36	psi " "
[$w = (LL\,soil + LL\,blast + DL)\times 16/144$ = $\begin{bmatrix}95\\95\end{bmatrix}\begin{bmatrix}283\\339\end{bmatrix}\begin{bmatrix}10\\10\end{bmatrix}$	43.1	49.3	pli TL]

Bearing Length: (of floor joist on top beam or column or wall)

Eq. 11

At each end of 2-span joist:
$L' = wL/(1.5\times 2.67\,F_{cL})$ =

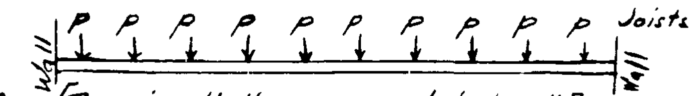
		Lower	Higher	
A. Fallout: $\begin{bmatrix}25.6\\28.3\end{bmatrix}\begin{bmatrix}66\\80\end{bmatrix}$	$\begin{bmatrix}190\\385\end{bmatrix}$	2.22"	~~1.47~~ USE 2"	A.
B. Combined: $\begin{bmatrix}43.1\\49.3\end{bmatrix}\begin{bmatrix}66\\80\end{bmatrix}$	$\begin{bmatrix}190\\385\end{bmatrix}$	3.74"	2.56"	B.

At midspan: $2\times L' = 2(wL)/(1.5\times 1.6\,F_{cL})$ = 7.41" 4.90" A.
[ ][ ] Same as above [ ] 12.48" 8.54 B.
[ ][ ] [ ]

USE NEXT HIGHER 1/2 IN.

Assume that in each column line, there is a column under each of the 2 end joists, ignoring joists resting their full length on the basement end walls.

Also, assume that column spacing is either at some whole number (integer) multiple of the joist spacing (usually 12, 16 or 24 in. c-c), or at an integer-divided-by-two multiple of the same. This means that all columns will be under joists; or that half or more will be, with the others located midway between joists. (An occasional irregularity in joist spacing should be treated so that the columns-per-joist ratio is not increased, but is decreased, over the predominating ratio.) If top beam must be discontinuous, break it between 2 joists, and put a column under each of the 2 joists, with top beam ends between joists and columns.

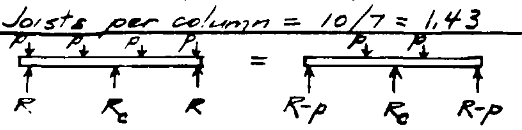


Case [Beam is vertically unsupported at walls]

① 

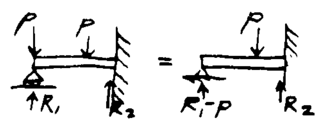
Columns @ 1½-joist spacing

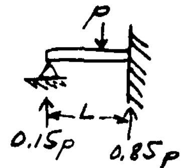
Joists per column = 10/7 = 1.43

(1A) 

Joists per column = 4/3 = 1.33
Looking @ shortest top beam case (of Case ①).

Use either half as a propped cantilever and design member: Ref. 1; p2-202 #14



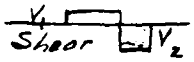


Net
$0.15p = V_1$ $= V_2$ (max. V)

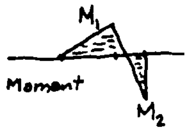
$$(\text{net})\ V_1 = \frac{p(L/3)^2}{2L^3}\left(\frac{2}{3}L + 2L\right) = \frac{4}{27}p = 0.15p$$

Ref. 1, p. 2-20, #14

$$R_2 = V_2 = \frac{p\left(\frac{2}{3}L\right)}{2L^3}\left(3L^2 - \left(\frac{2}{3}L\right)^2\right) = \frac{23}{27}p = 0.85p$$



$$R_1 - p = V_1 \quad \therefore R_1 = 1.15p$$
(in actual case)



$$M_1 = \frac{4}{27}p\left(\frac{2}{3}L\right) = \frac{8}{81}pL = 0.099\,pL$$

$$M_2 = p\left(\frac{2}{3}L\right)\left(\frac{L}{3}\right) \times \frac{\left(2L/3 + L\right)}{2L^2}$$

$$M_2 = pL \times \frac{2}{3} \times \frac{1}{3} \times \frac{1}{2} \times \frac{5}{3} = \frac{5}{27}pL = 0.185\,pL$$

EQ. NO.

FLEXURE:

$M_{max.} = 0.185\,pL = F_b \times \frac{bd^2}{6}$ [L = c-c. of supports] 22

HORIZONTAL SHEAR:

$V_{max.} = 0.85\,p = \frac{2}{3}F_v\,bd$ 23

V for design is taken at d distance from support center (no effect in this case, however).

BEARING LENGTH (on top and bottom of top beam):

Top of top beam: $L' = p/bF_{c\perp}$ 24

Bottom of top beam:

Propped end: $R_1 = 1.15p = bL'F_{c\perp}$ or $L' = \dfrac{1.15p}{bF_{c\perp}}$ 25

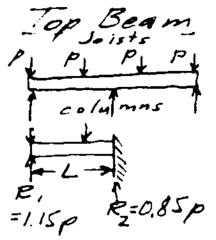
Fixed end: $R_2 = 0.85\,p = bL'F_{c\perp}$

This end is continous, so total bearing length at midspan is $2 \times L'$ (that is, L' each way from column support R_2)

$$2 \times L' = \frac{2 \times 0.85\,p}{bF_{c\perp}}$$ 26

Case 1A

Top Beam



Joist Spacing, j — Member Strength: Lower, Higher

Allowable top beam load per joist

p

Using Eq.

Triple 2x8's (1.5x7.25), edgewise, top beam

Eq.	Formula	Joist Spacing, j	Lower	Higher	
22	Flexure: $p = \frac{4.5 \times 7.25^2}{6} \times \frac{F_b \begin{bmatrix}925\\1750\end{bmatrix}}{0.185\,L}$	12"	10951	20717	lb.
	$L = 1.5 \times$ joist spacing	16"	8213	15538	"
		24"	5475	10359	"
23	Horizontal shear: $p = \frac{2}{3} \times \frac{4.5 \times 7.25}{0.85} \times F_v \begin{bmatrix}100\\140\end{bmatrix}$	12", 16", 24"	2559	3582	lb.
			Controlling.		

Note: It would be expected that horiz. shear would be controlling over flexure in such short, heavily loaded members.

Bearing length: (On top and bottom of top beam)*

Eq.	Formula	Lower	Higher
25	Propped end: (bottom) $L' = \frac{1.15\,p \begin{bmatrix}2559\\3582\end{bmatrix}}{4.5\,F_{c\perp}} =$	3.44"	2.38"
24	$\begin{bmatrix}190\\385\end{bmatrix}$ $\boxed{\text{Top of top beam } L' = \frac{p}{4.5F_{c\perp}} =}$	2.99"	2.07"
26	Overall, at continuous support: (bottom) $2L' = \frac{2 \times 0.85\,p}{4.5 \times F_{c\perp}} =$	5.09"	3.51"

Round to next higher 1/2 in.

Quadruple 2x6's (1.5x5.5), edgewise, top beam

(Flexure checked as not controlling)

Eq.	Formula	Joist Spacing, j	Lower	Higher
23	Horiz. shear: $p = \frac{2}{3} \times \frac{6 \times 5.5}{0.85} \times F_v \begin{bmatrix}100\\140\end{bmatrix}$	All j's	2588	3624

Bearing length: (As above)*

Eq.	Formula	Lower	Higher
25	Propped end: (bottom) $L' = \frac{1.15\,p \begin{bmatrix}2588\\3624\end{bmatrix}}{6\,F_{c\perp}} =$	2.61"	USE 2" min. ~~1.80"~~
24	$\begin{bmatrix}190\\385\end{bmatrix}$ $\boxed{\text{Top of top beam } L' = \frac{p}{6F_{c\perp}} =}$	2.27"	USE 2" min. ~~1.57"~~
26	Overall, at continuous end: (bottom) $2L' = \frac{2 \times 0.85\,p}{6\,F_{c\perp}} =$	3.86"	2.67"

Round to next higher 1/2 in.

* Use joist blocks and cap blocks (2 in. or thicker), as necessary, to provide required bearing lengths.

Columns for Upgrading (Sheet 1 of 3)

Try various column sizes, for added supports at floor joist midspans, using same lower and higher wood strengths as above with floor trial designs.
Assume 8 ft column lengths, pin-end conditions.

ESTIMATE OF NEEDS RANGE:

	Member Strengths Lower	Higher	
Joist spans (PC support conditions): c-c supports	66	80	in.
Contributory area to col., per joist on 16" centers: $jL/144 = 16L/144 =$	7.33	8.89	sf
Axial load per joist, unit [191+10; 213+10]	201	223	psf TL
" " " " , total	1473	1982	lb.
Colum DL (assuming 4x4 (3.5"x3.5") x 96" @ 40 pcf)	27	27	"
Axial load per joist, total, excluding top beam, if any, and wt. of column.	1446	1955	"

column load at normal stress F_c'

Eq. 2x4 column:

13 $L_1/d_1 = 96/1.5 = 64$ (controlling). Code max. ≤ 50. N.G.

2x4 column, lateral support @ mid-ht:

Eq.		Lower	Higher	
13	$L_1/d_1 = 48/1.5 = 32$ (controlling)	$E = 1.2 \times 10^6$	1.5×10^6	psi
	$L_2/d_2 = 96/3.5 = 27.43$	$F_c = 650$	1150	"
15	$K = 0.671\sqrt{E/F_c} =$	28.83	24.23	
		$L/d > K$		
17	$F_c' = 0.3E/\overline{32}^2 =$	352	439	psi
18	$P = 1.5 \times 3.5 \times F_c' =$	1848	2305	lb.

4x4 column:

Eq.		Lower	Higher	
13	$L/d = 96/3.5 = 27.43$ (as above)	E & F_c as above.		
15	K = same as above =	28.83	24.23	
16	$F_c' = F_c\left[1 - \frac{1}{3}\left(\frac{L}{dK}\right)^4\right] =$	$L/d < K$	$L/d > K$	
		472	—	psi
17	$F_c' = 0.3E/\overline{27.43}^2$	—	598	"
18	$P = 3.5 \times 3.5 \times F_c' =$	5782	7327	lb.

2x6 column:

13 $L_1/d_1 = 64$, and comments re 2x4 col. N.G.

2x6 column, supported @ mid-ht:

Eq.		Lower	Higher	
13	$L_2/d_2 = 96/5.5 = 17$ $L_1/d_1 = 48/1.5 = 32$ (controlling).	1.3	1.8×10^6	E, psi
		725	1250	F_c "
15	$K = 0.671\sqrt{E/F_c} =$	28.41	25.46	
		$L/d > K$		
17	$F_c' = 0.3E/\overline{32}^2 =$	381	527	psi
18	$P = 1.5 \times 5.5 \times F_c' =$	3143	4348	lb.

2x6 column, supported @ 1/3 & 2/3 ht.:

Eq.		Lower	Higher	
13	$L_2/d_2 = 17$ $L_1/d_1 = 32/1.5 = 21\frac{1}{3}$ (controlling)	E, F_c & K's as above.		
		$L/d < K$		
16	$F_c' = F_c\left[1 - \frac{1}{3}\left(\frac{L}{dK}\right)^4\right] =$	648	1045	psi
18	$P = 1.5 \times 5.5 \times F_c' =$	5346	8621	lb.

Columns for Upgrading (Sheet 2 of 3)

		Member Strength Lower	Member Strength Higher	
	E =	1.3	1.8×10^6	psi
	F_c =	725	1250	"
Eg. 2x8 column, supported @ midht:				
13 $L_2/d_2 = 96/7.25 = 13^+$ $L_1/d_1 = 48/1.5 = 32$ (controlling)				
15 K = same as 2x6		$L/d \geq K$		
17 F_c' = same as 2x6 (midht supp.) =		381	527	psi
18 $P = 1.5 \times 7.25 \times F_c'$ =		4143	5731	lb.
2x8 column supported @ 1/3 & 2/3 ht:		E, F_c & K's as above.		
Same as 2x6 w/ same supports, down to:		$L/d < K$		
16 F_c' = same		648	1045	psi
18 $P = 1.5 \times 7.25 \times F_c'$		7047	11364	lb.
4x6 column:		E, F_c & K's as above.		
13 $L_1/d_1 = 96/3.5 = 27.43$ (as above)		$L/d < K$	$L/d > K$	
16 $F_c' = F_c\left[1 - \frac{1}{3}\left(\frac{L}{dK}\right)^4\right]$ =		515	—	psi
17 $F_c' = 0.3E / 27.43^2$		—	718	"
18 $P = 3.5 \times 5.5 \times F_c'$		9914	13822	lb.
4x6 column, supported @ midht:		E, F_c & K's as above		
13 $L_1/d_1 = 48/3.5 = 13.71$ $L_2/d_2 = 96/5.5 = 17.45$ (controlling)		$L/d < K$		
16 $F_c' = F_c\left[1 - \frac{1}{3}\left(\frac{L}{dK}\right)^4\right]$ =		691	1158	psi
18 $P = 3.5 \times 5.5 \times F_c'$ =		13302	22292	lb.

Above are "normal" design values for column axial loads; for UPGRADING in host areas (blast free field $\leq$ 5 psi):

A. Fallout shielding only — soil live loads, for example:*
Increase axial load P values above by 15%, assuming load duration will be less than or equal to 2 mos.

B. Combined:* Fallout: Use soil load < A.; subtract actual soil load from 1.15 times column's normal, axial live load capacity, leaving a residual capacity for blast.
Blast: Using this residual capacity for blast, remove the 15% put in for soil loads (2 mos. or less), and substitute a 100% increase for impact load (of blast) — that is:

$$\left[\begin{matrix}\text{Residual} \\ \text{capacity for} \\ \text{blast}\end{matrix}\right] \times \frac{2}{1.15} = \text{Blast capacity (lb.)}$$

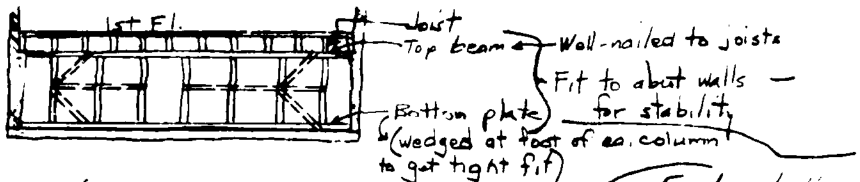
Divide Blast capacity (lb.) by column's contributory floor area served (Sheet 1), to get blast capacity (psf) — then divide by 144 to get it in psi.

(*) Saving the factor of safety (FS) of about 30% (or 1.30) for buckling failure insurance.

<u>Note</u> that "2x" columns cannot be longer than 75" [$L/d = 75/1.5 \leq 50$], unless they have lateral support in the weak (1½") direction. (See Eq. 13)

<u>Note</u> that lateral supports can be provided by 1"x4" or 1"x6" boards, well-nailed, flat on both sides of columns, OR by blocking between columns* — either one carried to <u>wedged</u> contact with end basement walls, OR by 1"x4" or 1"x6" boards, well-nailed, as diagonals from top to bottom plates, counter-directional with one someplace on each side of the columns.
Latter (diagonals) technique can be used to allow unobstructed opening between columns, wherever desired.

<u>NOTE</u>: Get a copy of latest (shirt-pocket) grading rules, and <u>re-grade</u> all wood members to be used in stress-graded situations.



<u>Example</u>:
Joists per column = 10/7 = 1.43

Fasten bottom plate with stud driver.

Always put column under last joist at each end (excluding one, or double, resting on each foundation wall)

Run <u>diagonals</u> on one side of each column line, at angle approx. 90° to those on other side of column line.
Lateral supports for columns may be <u>blocking</u>, <u>or</u> <u>face boards</u> on each side of column line — <u>made stable</u> by <u>diagonals</u>, <u>or</u> by <u>continuous blocking</u> through entire column line and <u>wedge-fitted</u> to both end walls

(*) Use of steel strapping is another alternative.

NOTATION [11]

A = area of cross-section (in.^{2})

b = breadth (edge width) of rectangular member (in.)

b′ = width of (floor) area supported (in.)

C = deflection coefficient (= L / maximum deflection)

C′ = beam formula constant

c = distance from neutral axis to extreme fiber (in.)

DL = dead load (psf)

d = depth of rectangular member (in.)

E = modulus of elasticity (psi)

F_b = design value for extreme fiber in bending (flexure) (psi)

F_c = design value for compression parallel to grain (psi)

F_c' = F_c, adjusted for L / d ratio (psi)

$F_{c\perp}$ = design value for compression perpendicular to grain (psi)

F_g = design value for end grain in bearing parallel to grain (psi)

F_v = design value in horizontal shear (psi)

I = moment of inertia (in.^{4})

j = joist spacing (in.)

K = for columns, largest slenderness ratio, L / d, where intermediate column formula applies (see Eq. 15 and 16)

L = span length of horizontal member (usually center-to-center of supports); or distance (c-c.) between column lateral support points (in.)

---

[11] Generally follows Reference [3].

NOTATION (concluded)

L′ = required bearing length in compression perpendicular to grain (in.)

LL = live load (psf)

M = bending or resisting moment, maximum (in.-lb)

P = total concentrated load or axial load (lb)

p = load (live and dead) <u>per joist</u> on top beam, post/stud, etc. (lb)

pcf = pounds per cubic foot

pli = pounds per linear inch

psf = pounds per square foot

psi = pounds per square inch

q = design strength (beam) for static loads (psi)

S = section modulus (= I / c) (in.^{3})

TL = total load (= DL + LL) (psf)

t = thickness (in.)

V = vertical design shear (= horizontal) in beams (lb)

W = total uniformly distributed load (lb)

w = uniformly distributed load per unit length (pli)

y = deflection of member, usually at mid-length (in.)

REFERENCES

1. Manual of Steel Construction, American Institute of Steel Construction, Inc., Wrigley Building (8th floor), 400 North Michigan Avenue, Chicago, Illinois 60611, 7th ed., 1970. (Eighth edition, 1980, was received October 17, 1980.)

2. "Design Values for Wood Construction," 1980 Supplement to National Design Specification for Wood Construction, 1977 edition [Reference 3]. Shirt-pocket size grading rules may be obtained from the following grading associations:

NELMA	Northeastern Lumber Manufacturers Association, Inc. 4 Fundy Road Falmouth, Maine 04105
NHPMA	Northern Hardwood and Pine Manufacturers Association, Inc. Northern Building Green Bay, Wisconsin 54301
NLGA	National Lumber Grades Authority (Canada) P. O. Box 97 Ganges, B.C., Canada VDS 1E0
RIS	Redwood Inspection Service 1 Lombard Street San Francisco, California 94111
SPIB	Southern Pine Inspection Bureau 4709 Scenic Highway Pensacola, Florida 32504
WCLIB	West Coast Lumber Inspection Bureau 6980 S.W. Varnes Road P.O. Box 23145 Portland, Oregon 97223
WWPA	Western Wood Products Association 1500 Yeon Building Portland, Oregon 97204

3. National Design Specification for Wood Construction, 1977 edition (Structural Lumber, Glued Laminated Timber, Timber Pilings, Fastenings), National Forest Products Association, 1619 Massachusetts Avenue, N.W., Washington, D.C. 20036.

4. Murphy, H. L., Upgrading Basements for Combined Nuclear Weapons Effects: Predesigned Expedient Options, Stanford Research Institute[12] Technical Report for Defense Civil Preparedness Agency,[13] October 1977.

5. Gaylord, E. H., Jr., and C. N. Gaylord, editors, Structural Engineering Handbook, McGraw-Hill, 1968.

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[12] Now SRI International

[13] Now Federal Emergency Management Agency

Appendix D1

BLAST-RESISTANT DESIGN/ANALYSIS OF STEEL MEMBERS

CONTENTS

AD-A097 915 SRI INTERNATIONAL MENLO PARK CA F/G 13/13
UPGRADING BASEMENTS FOR COMBINED NUCLEAR WEAPONS EFFECTS: PREDE--ETC(U)
JUL 80 H L MURPHY DCPA01-77-C-0227
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TABLES

FIGURE

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Appendix D1

BLAST-RESISTANT DESIGN/ANALYSIS OF STEEL MEMBERS[1]

Introduction

It is expected that the reader/user of this appendix will have first read Appendix A containing information applicable to blast-resistant design/analysis generally. This appendix provides information specific to such design/analysis using steel plates and rolled shapes.

As before (Appendix A), useful material developed by Newmark [1][2] is included in quotes below (parenthetic insertions or connective words are by this writer).

Design in Steel

"(Blast-resistant) designs in structural steel . . . are based on plastic design principles. For members responding primarily in flexure, if the sections are sufficiently ductile to permit redistribution of moment after the first inelastic action begins, the yield moment is taken as the fully plastic moment of the cross-section.

"Structural carbon steel, i.e., ASTM A-7, A-36, or A-373, possesses a high degree of ductility and strain hardens markedly. Therefore, the zones of inelastic behavior in structural elements formed of these alloys will be widespread. A ductility factor (μ) in flexure of 10 has been used for design in these alloys.

"Higher strength steels . . . have less ductility and strain harden less than the above mild steels. The zones of inelastic behavior will be more limited in extent with limited plastic hinge rotational capacity. Therefore, a ductility factor of no more than 3 is recommended for designs of flexural members in these materials.

"Compression members should be designed for a ductility factor of 1.3. Tension members may be designed with the ductility factor appropriate for the material in flexure.

---

[1] Flat Plate/Sheet, Corrugated, and Builtup Sections.

[2] Brackets are used herein to indicate sources in the References list at the end of this appendix.

Stability Limitations

"In order that the yielded cross-section of a member continue to transmit the fully plastic moment through large rotations of the plastic hinge, limitations are placed on the cross-sectional dimensions to insure stability against buckling. Two types of instability are important for flexural members: (1) local buckling of elements of the section, and (2) lateral buckling of the compression flange. Lateral buckling is not commonly a problem for (floor or) door elements since the compressive flanges will ordinarily have continuous lateral support. However, in instances where compression flanges are (laterally) supported, the stability of the member should be evaluated following the procedures given in Reference [2]. . . . It is noted that tests have shown a lessened tendency towards buckling for rapid load application; therefore, the usual specified minimum values for yield stress may be used as the required stress level in equations for stability. . . .

"The following limitations on the dimensions of ASTM A-7, A-36, or A-373 steel sections are taken from Reference [2]. The relations are based on an axial yield stress level of 33 ksi, but tests have shown that sections meeting these requirements perform satisfactorily under rapid loading in spite of the increased yield point.

"<u>Stability Requirement for A-7, A-36 and A-373 Steel Sections</u>

Compression flange of WF, I or H section	$b/t_f \leq 17$
Web in shear	$d/w \leq 43$

where:

b = flange width (in.)

d = depth of section (in.)

t_f = flange thickness (in.)

w = web thickness (in.)

"Interaction expression for more complex stress states are given in Reference [2] which also contains more general expressions which must be used to set stability limitations on the dimensions of sections fabricated of higher strength steel.

"The structural resistance is defined by the yield level and the ductility. The product of these quantities is a measure of the energy absorption capacity of the structure. Continuity of structural elements provides an increased energy absorption capacity and should be provided where practicable in metal blast-resistant structures. To attain continuity, joints should be designed to develop the full flexural or axial capacity of the member; otherwise, deformations will be concentrated at the joints and the overall ductility of the element will be reduced.

"Welded construction can readily provide structural continuity. However, approved welding procedures, good weld and fabrication details, properly selected welding rods, and weldable base metal are essential if brittle response is to be avoided. Riveted or bolted joint details should be free of sheared edges and punched holes, and adequate edge distances should be provided.

Structural Steel Material Properties

"The (dynamic) design stresses (Table D1-1) are based on the yield strengths for the loading rate range expected in protective construction (References [1,3]).

Resistance Expressions

"The equations involved in the evaluation of the resistance of structural (steel) elements . . . are presented in . . . Table D1-2. These expressions define the flexural resistance, shearing resistance, (effective) natural period (of vibration) in flexure, and (centerline) yield deflection. The design stresses to be used with these expressions are in Table D1-1."

Table D1-1A

DESIGN STRESSES FOR STEEL

Steel	Axial Stress f_{dy},ksi	Shearing Stress v_{dy},ksi	Allowable Bearing Stress: Single Shear f_{by},ksi	Allowable Bearing Stress: Double Shear f_{by},ksi
Structural Carbon,[1] ASTM A-7, A-36, or A-373	42	25		
Corrugated Iron[2]	34	20		
Welds	42	29		
Rivets ASTM A-141	40	30	60	80
ASTM A-195	60	40	80	80
Bolts ASTM A-307	32	19	40	40
ASTM A-325	50	30	60	60

[1] For higher strength structural steels, use an axial design stress, f_{dy}, equal to the smaller of 1.10 times the specified minimum yield or 0.90 times the specified minimum ultimate strength. For design shearing stress, v_{dy}, use 0.60 f_{dy}.
(See facing page.)

[2] The value of f_{dy} has been selected to be used with a plastic modulus, Z, of 1.5 times the section modulus, S.

Source: Ret. 1, p. 153

Table D1-1B

Steel	Yield Tens.* ksi	Ult. Tens.* ksi	Axial Stress f_{dy}, ksi	Shearing Stress v_{dy}, ksi
Carbon:				
A529	42	60-85	50	25
High-Strength Low-Alloy:				
A242 (to 3/4" th. incl.)	50	70	55	33
A440 "	"	"	"	"
A441 "	"	"	"	"
A572 (to & incl. th.":)				
Grade 42 (4")	42	60	46	28
" 45 (1½")	45	60	50	30
" 50 (1½")	50	65	55	33
" 55 (1½")	55	70	61	36
" 60 (1")	60	75	66	40
" 65 (½")	65	80	72	43
A588 (to 4" th. incl.)	50	70	55	33

*Source: Manual of Steel Construction, 7th edition, 1970, American Institute of Steel Construction, Chicago; p. 5-212 thru -216.

NOTATION FOR TABLE D1-2

α = L_s/L_ℓ (design as 1-way in short direction for $\alpha < \frac{1}{2}$)

A = total area of element, in^2

A/b = A per inch width, in.

b = width of element of section, in.

d = depth of structural element, in.

E = elastic Youngs modulus, psi

f_{dy} = dynamic tensile yield stress, psi

I = moment of inertia of element, in^4

I/b = I per inch width, in^3

$k_1 = (77 + 180\,\alpha^3)\,\dfrac{Et^3}{12L_s^{\,4}}$, psi/in

$k_2 = (307 + 500\,\alpha^3)\,\dfrac{Et^3}{12L_s^{\,4}}$, psi/in

$K_\alpha^{\,2} = 3 - 2\alpha\sqrt{\alpha^2 + 3} + 2\,\alpha^2$

L = 1-way plate span, in.

L_ℓ = 2-way long span, in.

L_s = 2-way short span, in.

S = section modulus of element, in^3

S/b = S per inch width, in^2

t = plate thickness, in.

t_w = total web thickness of element, in.

v_{dy} = dynamic shearing yield stress, psi

W = plate weight, psf

Z = plastic modulus of section, in^3

Z $\cong$ 1.5 S for corrugated plate
1.15 S for I or WF section

Z/b = Z per inch width, in^2

Source: Ref. 1, p. 158

Table D1-2

RESISTANCE EXPRESSIONS FOR STEEL STRUCTURAL ELEMENTS

	Flexure q_y, psi	Shear q_y, psi	Period in Flexure T, sec.	Yield Defl. at ℄, x_y in
FLAT PLATE SECTION				
1-way Simple Support	$2f_{dy}\left(\frac{t}{L}\right)^2$	$2v_{dy}\left(\frac{t}{L}\right)$	$9.4\times10^{-3}L^2\sqrt{\frac{W}{Et^3}}$	$\frac{5}{32}\frac{q_y L^4}{Et^3}$
Fixed Support	$4f_{dy}\left(\frac{t}{L}\right)^2$	$2v_{dy}\left(\frac{t}{L}\right)$	$4.1\times10^{-3}L^2\sqrt{\frac{W}{Et^3}}$	$\frac{q_y L^4}{25.6\ Et^3}$
2-way Simple Support	$6f_{dy}\left(\frac{t}{K_\alpha L_s}\right)^2$	$2v_{dy}\frac{t}{L_s}\left[\frac{2}{3}(1+\alpha)\right]$	$2.2\times10^{-2}\sqrt{\frac{W}{k_1}}$	$\frac{q_y}{k_1}$
Fixed Support	$12f_{dy}\left(\frac{t}{K_\alpha L_s}\right)^2$	$2v_{dy}\frac{t}{L_s}\left[\frac{2}{3}(1+\alpha)\right]$	$1.9\times10^{-2}\sqrt{\frac{W}{k_2}}$	$\frac{q_y}{k_2}$
CORRUGATED SECTION				
1-way Simple Support	$\frac{8f_{dy}}{L^2}\frac{Z}{b}$	$\frac{2v_{dy}}{L}\frac{A}{b}$	$2.7\times10^{-3}L^2\sqrt{\frac{W}{E}\frac{b}{I}}$	$\frac{5}{384}\frac{q_y L^4}{E}\frac{b}{I}$
Fixed Support	$\frac{16f_{dy}}{L^2}\frac{Z}{b}$	$\frac{2v_{dy}}{L}\frac{A}{b}$	$1.2\times10^{-3}L^2\sqrt{\frac{W}{E}\frac{b}{I}}$	$\frac{q_y L^4}{207\ E}\frac{b}{I}$
BUILT-UP SECTION				
1-way Simple Support	$\frac{8f_{dy}}{L^2}\frac{Z}{b}$	$2v_{dy}\frac{dt_w}{bL}$	$2.7\times10^{-3}L^2\sqrt{\frac{W}{E}\frac{b}{I}}$	$\frac{5}{384}\frac{q_y L^4}{E}\frac{b}{I}$
Fixed Support	$\frac{16f_{dy}}{L^2}\frac{Z}{b}$	$2v_{dy}\frac{dt_w}{bL}$	$1.2\times10^{-3}L^2\sqrt{\frac{W}{E}\frac{b}{I}}$	$\frac{q_y L^4}{307\ E}\frac{b}{I}$

Source: Ref. 1, p. 159

Applications

This section is limited to use of steel sheets and plates as barriers, especially as closures to prevent exterior air blast from entering the basement shelter. However, data useful for other steel applications, such as use of corrugated steel sheets, builtup sections, welds, rivets and bolts, are included elsewhere herein.

It is assumed that the reader/user has reviewed Appendix A herein.

Local availability of steel sheets, plates, and shapes is discussed below in Appendix E1.

A. Allowable Stresses

Allowable stresses for blast-resistant design are shown in Table D1-1. For design of sheet/plate closures, one needs a value for f_{dy} for the particular available steel planned for use, based on its specific ASTM specification number. The f_{dy} value should be in psi (ksi value times 1000 equals the psi value), and is used for both the axial and flexural allowable stresses. (Allowable shear stress v_{dy} is not needed for the applications contemplated herein.)

For an illustrative example: assume A36 steel for use; f_{dy} is 42 ksi (Table D1-1A) or 42,000 psi.

B. One-way Flat Plate Closures, Simple Supports

This design situation applies when the closure sheet or plate is simply supported (i.e., not clamped) on two opposite edges only. The applicable basic formula is the first one of Table D1-2; q_y is found, using known values of plate thickness t (in.) and the clear span L (in.) between supports. Also used is the formula relating p_{dm} (peak value of the blast pressure that can be resisted by the steel plate closure, psi) to q_y and the ductility factor μ.[3]

Graphic solutions for these combined (two) formulas, for A7/A36/A373 steels in thicknesses of 16 gage (0.06 in.) and 1/8" to 3/4", are shown in Figure D1-1, which is accompanied by a table of correction factors that is discussed further below.

Continuing our example: for 3/4" th. plate and clear span L of 50 in., read p_{dm} = 18 psi in Figure D1-1. (Curves of Figure D1-1 are ended arbitrarily at peak p_{dm} value of 50 psi and peak plate weight of about 500 pounds (for square plate of length L on each side).)

---

[3] See third paragraph of earlier section on "Design in Steel" where μ = 10 is recommended for carbon steels and μ = 3 for higher strength steels.

CORRECTION FACTORS FOR FIGURE D1-1

For Carbon Steels

ASTM-	Factor
A7	1.000
A36	1.000
A373	1.000
A529	1.190

For High-Strength Steels

ASTM-		Factor
A242		1.149
A440		1.149
A441		1.149
A572		
Grade	42	0.961
"	45	1.044
"	50	1.149
"	55	1.274
"	60	1.378
"	65	1.504
A588		1.149

For Two-way Plates

Ratio of longer to shorter clear spans, L_l / L_s	Factor
1.0	3.000
1.1	2.736
1.2	2.531
1.3	2.366
1.4	2.232
1.5	2.121
1.6	2.028
1.7	1.948
1.8	1.879
1.9	1.820
2.0	1.768
> 2.0	1.000

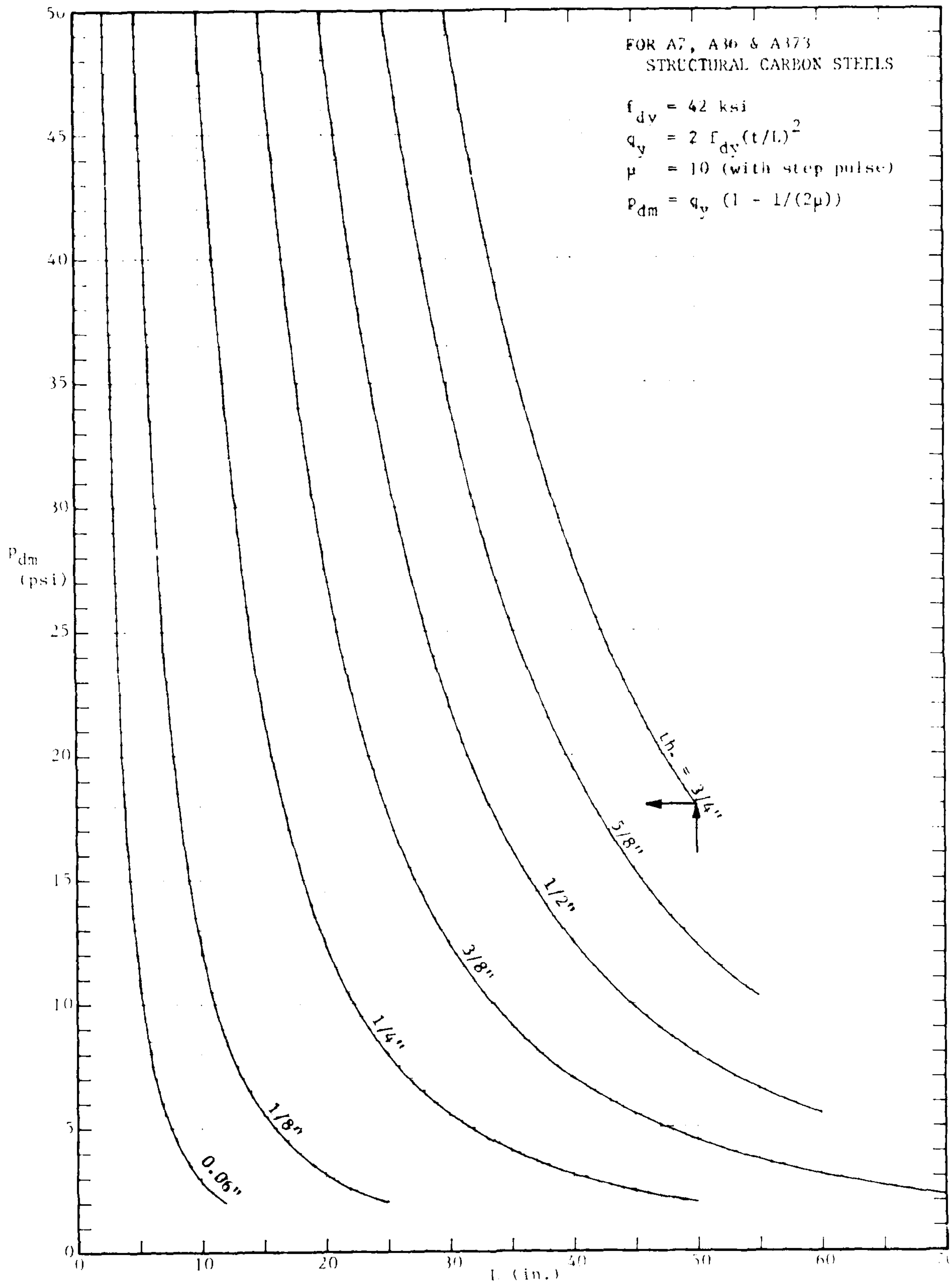

Figure DI-1 STEEL PLATE/SHEET DESIGNS (One-way, Simple Supports)

C. Correction Factors for Other Steels

Correction factors for use with Figure D1-1 are shown in a table accompanying the figure; they allow use of all other steels listed in Table D1-1, each with the appropriate ductility factor μ as discussed above. The figure is used as described just above, then the p_{dm} value obtained is multiplied by the correction factor for the steel used.

Continuing our example: for the same plate and span, but this time with A529 steel: p_{dm} of 18 psi is multiplied by the correction factor of 1.190 for A529 steel, giving a corrected p_{dm} = 18 x 1.190 = 21.4 psi.

D. Correction Factors for Two-way Plates

Correction factors for use with Figure D1-1 are shown in the same accompanying table, and cover use of a two-way plate simply supported on all four edges, instead of the one-way simply supported plate that is the basis for the Figure. These correction factors vary with the ratio of longer clear span to shorter clear span, or L_L/L_s.

Continuing our example: using a two-way simply supported plate instead of one-way, shorter and longer clear spans are 50 and 60 in., respectively; use the shorter span with Figure D1-1 and read the same 18 psi for p_{dm}; A529 steel dictates using a correction factor of 1.190 (as above); correcting for two-way versus one-way plates inserts another correction factor, that for L_L/L_s = 60/50 = 1.2, or a factor of 2.531 from the table accompanying the Figure. Putting this together: p_{dm} = 18 x 1.190 x 2.531 = 54.2 psi.

NOTATION

(Excluding notation defined there and used in Tables D1-1 and D1-2)

b = flange width (in.)

d = depth of section (in.)

ksi = kips (kilo-pounds) per square inch (1 kip = 1,000 psi)

psi = pounds per square inch

p_{dm} = peak air blast pressure applied to member, psi

t_f = flange thickness (in.)

w = web thickness (in.)

μ = x_m/x_e = the ductility factor (see Appendix A)

REFERENCES

1. Newmark, N. M., Design of Openings for Buried Shelters, Report 2-67, U.S. Army Engineer Waterways Experiment Station, Vicksburg, Mississippi, July 1963; Chapter 7.

2. Commentary on Plastic Design in Steel, Manual of Engineering Practice No. 41, American Society of Civil Engineers (ASCE), New York, New York, 1961.

3. Anderson, F. E., Jr., R. J. Hansen, H. L. Murphy, N. M. Newmark, and M. P. White, Design of Structures to Resist Nuclear Weapons Effects, ASCE Manual of Engineering Practice No. 42, 1961, and Supplement 1963 (combined into a "1964 Edition").

Appendix E1

STRUCTURAL STEEL LOCAL AVAILABILITY AND USE FOR BLAST SHELTER UPGRADING

By: Ellis E. Pickering, P.E.
Senior Civil Engineer

CONTENTS

TABLES

Appendix E1

STRUCTURAL STEEL LOCAL AVAILABILITY AND USE FOR BLAST SHELTER UPGRADING

Introduction and Purpose

Among the various materials useful for construction of closure systems for blast shelters and for strengthening the floor over such shelters, are the standard structural steel products including plates, structural shapes including "I" beams and channels, structural tubing, and pipe.

These materials are those normally used in the structural framing for buildings, bridges, industrial plants, and for other heavy construction purposes. Structural steel materials are stocked rather widely on a regional and local basis.

Structural steel materials may be used for "expedient" or "engineered" upgrading of potential blast shelters. Usually such uses will involve provision of closure or strengthening of floor systems, or most likely both. Structural steel materials will be particularly useful, and in many cases required, for provision of the higher levels of blast protection, or where closure or floor spans are long.

Design guidance for the use of structural steel products in both expedient and engineered blast shelter upgrading is given elsewhere in this report (main text and Appendix D1). The purpose of this appendix is to describe the general characteristics and uses of these materials and to identify sources of supply, together with typical regional and local availabiity.

The reader desiring more detailed information on structural steel should refer to standard references [e.g.,1],[1] or consult with a Professional Civil or Structural Engineer. Actual designs should be prepared by professional engineers if time is available.

Shapes, Sizes, and Designations

A. Introduction

Structural steel materials considered include plate, rolled shapes, pipe and structural tubing, sheet piling, and railroad rail. Standard shapes, sizes, and designations are discussed for each in the following paragraphs.

---

[1] Brackets are used herein to indicate sources in the References list at the end of this appendix.

B. Structural Steel Plate

Structural steel plate is manufactured in a variety of thicknesses, widths, and lengths. Materials are designated as strip, bar, or plate depending on the relationship of thickness to width. For the purposes of the present applications, the useful materials are limited to the bar and plate designations of 1/4" thickness or more, which are grouped in thickness and width classifications as follows: Nominal thicknesses (in.) are 1/4, 5/16, 3/8, 7/16, 1/2, 5/8, 3/4, and 1, within which widths of 3-1/2" to 8" are bars and over 8" are plates.

C. Rolled Structural Shapes

Hot rolled structural steel shapes include products formerly referred to as "I" Beams, "Wide Flange" or "WF" Beams, "H" Beams, Channels, and Tees. The present standard designation system (together with the former system) and size ranges are given in Table E1-1.

The designation system shown in Table E1-1 is a method of identification for shapes and sizes of rolled structural steel shapes, employing a standard nomenclature. Its primary use is as a form of abbreviation for identification on drawings, but it is also recognized throughout the trade including sources of supply. The present designation includes three items: (1) type, shape, or group symbol; (2) nominal shape, depth

and (3) the weight in pounds per linear foot (except for angles, where the thickness of the metal is given instead).

The designation system was changed in 1970 [1]. Both the current and former designation systems are given in Table E1-1 since older tables and drawings are still in use.

---

[2] Actual dimensions differ significantly from nominal dimensions for some shapes. See the last section of this appendix for further explanation.

Table E1-1

ROLLED STEEL SHAPES - DESIGNATIONS AND SIZES

Generic Type or Group	Example Designation		Size Ranges	
Beams and Columns	Present	Former	From	To
Wide Flange Beams and Columns	W8x40	8WF40	W8x20	W36x300
Standard "I" Beams	S8x23	8I23	S3x7.5	S24x120
Light Wide Flange Columns	W6x20	6WF20	W4x13	W6x25
Light Beams	W10x15	10B15	W6x16	W16x31
Miscellaneous Columns	M6x25	6M25	M4x13	M8x34.3
Junior Beams	M10x9	10Jr9	M6x4.4	M12x11.8
Miscellaneous Shapes	M8x20	8M20	M8x22.5	M10x29.1
"H" Shaped Bearing Piles	HP12x74	12BP74	HP12x63	HP14x117
Structural Channels				
Standard Channels	C10x30	10[30	C3x6	C15x50
Miscellaneous Channels	MC12x45	12x4[45	MC8x18.7	MC18x58
Junior Channels	MC10x8.4	10Jr[8.4	MC10x6.5	MC12x10.6
Structural Angles				
Equal Leg Angles	L5x5x3/4	∠5x5x3/4	L3x3x3/16	L8x8x1-1/8
Unequal Leg Angles	L6x4x3/4	∠6x4x3/4	L3x2x3/16	L9x4x1
Structural Tees				
Cut from Wide Flange Shapes	WT8x25	ST8WF25	WT4x8.5	WT18x150
Cut from Standard "I" Beams	ST6x25	ST6I25	ST3x6.3	ST12x60
Cut from Miscellaneous Shapes	MT5x10.5	ST5M10.5	MT4x8.5	MT5x14.6
Cut from Light Beams	WT7x13	ST7B13	WT3x4.3	WT8x15.5
Cut from Junior Beams	MT6x5.9	ST6Jr5.9	MT3x2.2	MT6x5.9

Examples of the full description of products under the present and former systems follow:

W8x40 - A Wide Flange Beam or column (W) of 8" depth and weighing 40 pounds per foot.

S8x23 - A standard "I" Beam (S) of 8" depth and weighing 23 pounds per foot.

C10x30 - A standard Channel (C) of 10" depth and weighing 30 pounds per foot.

L5x5x3/4 - An equal leg Angle (L) of 5"x5" dimensions (outer faces) and 3/4" thickness.

ST6x25 - A structural Tee cut from a standard "I" Beam (S) of 6" depth and weighing 25 pounds per foot.

Certain special structural shapes have not been included in Table E1-1 because of their limited availability. These include special car building and ship building Channels, bulb Angles, Zees, and small rolled Tee Beams. Sizes and shapes of the car building and ship building Channels are given in Reference [1]. Details on other special shapes can be found in manufacturers' catalogs.

D. Pipe and Structural Tubing

Steel pipe (circular section) is manufactured in standard sizes ranging from 1/8" to 12" nominal diameter in standard, extra stong, and double extra strong weights. The nominal diameter is an approximation of the inside diameter of the pipe in the larger sizes of standard weight. The extra strong and double extra strong weights have reduced inside diameters corresponding to the increased wall thicknesses.

Dimensions of selected sizes and weights of steel pipes are given in Table E1-2.

Structural tubing (square or rectangular section) is manufactured in sizes up to 12" in major dimension. Selected sizes and weights are given in Table E1-3.

Table E1-2

SELECTED PIPE SIZES - DIMENSIONS AND WEIGHTS

Nominal Diameter (in.)	Outside Diameter (in.)	Inside Diameter (in.)			Wall Thickness (in.)			Weight Per Foot (lb)		
		S*	ES	DES	S	ES	DES	S	ES	DES
3	3.500	3.068	2.900	2.300	0.216	0.300	0.600	7.58	10.25	18.58
4	4.500	4.026	3.826	3.152	0.237	0.337	0.674	10.79	14.98	27.54
5	5.563	5.047	4.813	4.063	0.258	0.375	0.750	14.62	20.78	38.55
6	6.625	6.065	5.761	4.897	0.280	0.432	0.864	18.97	28.57	53.16
8	8.625	7.981	7.625	6.875	0.322	0.500	0.875	28.55	43.39	72.42
10	10.750	10.020	9.750	8.750	0.365	0.500	1.000	40.48	54.74	104.13
12	12.750	12.000	11.750	10.750	0.375	0.500	1.000	49.56	65.42	125.49

* Designation: S = Standard Weight

ES = Extra Strong Weight

DES = Double Extra Strong Weight

Table E1-3

STRUCTURAL TUBING SIZES - DIMENSIONS AND WEIGHTS

Square Section						Rectangular Section					
Outside Dimensions (in.)	Weight Per Ft (lb) For Various Wall Thicknesses (in.)					Outside Dimensions (in.)	Weight Per Ft (lb) For Various Wall Thicknesses (in.)				
	1/2	3/8	5/16	1/4	3/16		1/2	3/8	5/16	1/4	3/16
12x12	74.54	57.23	48.24	39.03	N/A*	12x6	54.15	41.93	35.49	28.83	N/A
10x10	60.95	47.03	39.74	32.23	N/A	10x6	47.35	36.83	31.24	25.44	N/A
8x8	47.35	36.83	31.24	25.44	N/A	8x6	40.55	31.73	26.99	22.04	16.85
7x7	40.55	31.73	26.99	22.04	16.85	8x4	34.48	27.04	23.02	18.82	14.41
6x6	34.48	27.04	23.02	18.82	14.41	7x5	34.48	27.04	23.02	18.82	14.41
5x5	27.68	21.94	18.77	15.42	11.86	6x3	N/A	19.39	16.65	13.72	10.58
4x4	N/A	16.84	14.52	12.02	9.31	5x3	N/A	16.84	14.52	12.02	9.31

* N/A = Not available

E. Other Structural Steel Materials

The materials listed in the previous paragraphs are those generally used in steel building framing, bridges, and industrial applications. In addition to these materials, other structural steel materials may be found to be useful. These include railroad rails and sheet steel piling.

Railroad rails in either a new or used condition offer considerable strength. Most rail is rolled to an American Society of Civil Engineers (ASCE) or an American Railway Engineering Association (AREA) pattern and is classified on both a pattern and weight per yard basis. Table E1-4 lists the properties of selected rail sections.

Sheet steel piling may be available near the coasts and in inland areas near lakes and rivers. It is rolled in several patterns with interlocking edges. Patterns include straight web, arch web, and Zee. Properties of typical sections are given in Table E1-5.

Table E1-4

TYPICAL RR RAIL SECTIONS - DIMENSIONS AND WEIGHTS*

Weight Per Yard (lb)	Height (in.)	Base Width (in.)	Head Width (in.)#
155	8	6-3/4	3
140	7-5/16	6	3
132	7-1/8	6	3
115	6-5/8	5-1/2	2-23/32
100	6	5-3/8	2-11/16
75	4-13/16	4-13/64	2-15/32

* New Condition

\# Maximum at bottom of head

Table E1-5

TYPICAL SHEET PILE SECTIONS - DIMENSIONS AND WEIGHTS

Section	Nominal Width (in.)	Depth (in.)	Thickness Web (in.)	Thickness Flange (in.)	Weight Per Ft (lb)
Straight Web	15	-	1/2	-	40
	15	-	3/8	-	35
Arch Web	19-5/8	3-1/4	3/8	3/8	36
	16	1-11/32	1/2	1/2	37.3
	16	1-11/32	3/8	3/8	30.7
Deep Arch Web	16	6	31/64	3/8	42.7
	16	5	3/8	3/8	36
Zee	21	11-1/2	3/8	1/2	56
	18	12	3/8	1/2	57
	18	12	3/8	3/8	40.5

Material Strength Grades

Structural steel is produced in various grades related to strength and weldability as prescribed in American Society for Testing and Materials (ASTM) specifications. Applicable specifications include:

ASTM A7	Steel for Bridges and Building
ASTM 36	Structural Steel
ASTM A242	High Strength Low Alloy Structural Steel
ASTM A373	Structural Steel for Welding
ASTM A440	High Strength Structural Steel
ASTM A441	High Strength, Low Alloy Structural Manganese Vanadium Steel

The principal variation among the types relates to strength as indicated by yield point. All are weldable except A440, which is not recommended for welded use. The yield point specification values (psi) are:

A7	33,000
A36	36,000
A242	42,000
A373	33,000
A440	42,000
A441	42,000
Rail Steel	39,000

The various standard products are generally produced in grades as follows:

Bar and Plate: A7, A36, A242, A373, A440, A441

Pipe: A7, A36

Railroad Rail: AREA Specification

Rolled Structural Shapes: A7, A36, A242, A373, A440, A441

Sheet Pile: A7, A36

Structural Tubing: A7, A36

Principles of Use and Application

A. General Principles

Structural steel materials are capable of providing relatively high degrees of blast-resistant upgrading as compared to wood; however, they present more difficult problems in fabrication and installation. The principle problems are cutting to size, plus fastening of individual members to each other and to the the basic shelter space structure. The methods of fabrication and installation will vary somewhat between expedient and engineered upgrading applications because of available time and skills.

a. Expedient Applications

The expedient upgrading process will be limited by available time and the skills of the work force. In this process the following general principles should be applied:

(1) The structure (closure, beam support, etc.) should be designed around the material readily available locally.

(2) All cutting to size should be accomplished at the point of supply, where flame cutting, sawing, or shearing may be readily accomplished. On the job cutting would present extreme difficulty, although rough oxyacetylene cutting may be possible if tools and skills are available.

(3) Fastenings should be restricted to simple bolting wherever possible. Bolt holes may be drilled locally although it also would be more efficient to have this done at the point of supply. Fastenings should be designed so as not to be stressed by the blast wave, i.e., the blast wave should add no tension or shear load to the fastenings.

(4) The size and weight of individual structural members should be limited to that which can be handled and erected by a few men without the use of heavy materials-handling equipment.

(5) Advantage should be taken of crisis build-up time to procure pre-cut and pre-drilled material, and to store it for immediate use in the proposed shelter space.

(6) Material strength grades should be limited to A7 or A36 grades wherever possible because of the greater ease in cutting, drilling, or welding.

b. Engineered Applications

The engineered application process permits the utilization of a normal time-frame for design, fabrication, and installation (where desired). The design should be accomplished by a Professional Civil or Structural Engineer and the conventional procurement, fabrication, and installation process should be followed. In this case:

(1) Advantage may be taken of higher strength materials.

(2) More sophisticated fastening systems may be employed to obtain material economy through the use of moment-resisting joints and other economic design features.

(3) Structural welding of subassemblies may be employed, and final erection may also utilize welding, if the structure is to be installed immediately. If the parts of subassemblies are to be stockpiled, however, certain principles listed above for expedient applications - i.e., those related to size, weight, and bolting - should be followed. Such stockpiling is applicable where immediate installation might interfere with the normal use of the proposed shelter space.

B. Potential Applications

Potential applications of structural steel materials in blast shelter upgrading are listed in Table E1-6. For specific design data, member sizing, and blast resistance, see Appendix D1.

Local Availability

A. Manufacturing and Distribution System

Structural steel products are manufactured on a regional basis with major production centers (mills) located in the Ohio Valley, Great Lakes region, Eastern Seaboard (Baltimore), Southeast (Alabama), Rocky Mountain region (Utah and Colorado), and California. In addition, imports are available generally at seaboard locations. The distribution system is from mill to regional warehouses, or to combination warehouse-fabrication yards, with either type owned by the major structural steel producers or large independents. The mill inventory also serves as a regional stock.

The regional network is supplemented by a system of local warehouses and small fabricators, usually of an independent nature. The local facilities obtain their supplies from regional warehouses or direct from mills for large o[illegible]s.

Table E1-6

POTENTIAL APPLICATIONS OF STRUCTURAL STEEL MATERIALS TO BLAST SHELTER UPGRADING *

Material	Potential Application	Application Notes
Structural Steel Plate	Closure of small openings such as doors, windows, ventilation shafts, utility penetrations in floors, area ways, etc.	Provide hold-down by bolting, loading with sand bags, bracing, etc.
	Covering for structural steel frame closure for large openings.	
Rolled Structural Steel Shapes		
Light-Weight Channels and "I" Beams	Closure of openings in flat-wise position	Tie together by tackwelding, transverse bolting through flanges, etc. Provide hold-down by bolting, loading with sand bags, bracing, etc.
	Covering for main structural steel frame closure for large openings	
	Framing to support plate covering for small openings	

* Special attention must be given to attachment of closure and support systems to the structural framework of the building under consideration. In many cases the framework will require upgrading.

Table E1-6 (concluded)

Wide-Flange Beams, "H" Piles, "I" Beams, Channels, Angles, and Tee Beams	Structural frame to support closures for large openings such as vehicle doors, stairwells, and elevator shafts. Covering may include steel plate, wood planks, stressed skin plywood panels, etc.	Fastenings can be made by welding, drilling, and bolting, etc. Rather precise cutting is required
	Columns to strengthen existing columns; or added columns to reduce spans in beams, girders, and slabs.	Requires rather precise cutting to length, or wedging. Top and bottom bearing plates are required.
<u>Pipe and Structural Tubing</u>	Columns (see above).	(See above).
	Rectangular tubing can be used in flat-wise position for direct closure of openings or for covering structural steel frames for large openings.	Tie together by tackwelding. Provide hold-down by bolting, covering with sand bags, etc.
	Rectangular tubing may be used for fabrication of a structural frame for closure of large openings.	
<u>Other Structural Steel Materials</u>		
Railroad Rails	Closure of openings in base-to-base position.	See notes for flat-wise application.
	Columns	See column notes above.
Sheet Pile	Direct closure of openings	Lock edges together.
	Covering for structural steel frame for large openings.	Provide hold-down by tack-welding, bolting, loading with sand bags, etc.

Imported materials go to the independent regional establishments or to large volume local dealers.

Except in the least populated areas of the country, a regional or local warehouse may be expected to be found within one-day truck distance (say 100 miles) of any population center.

All establishments have capability of some fabrication, the minimum being drilling and cutting to size (flame cutting, shearing or sawing). Most also have welding and grinding facilities for prefabrication to some degree. These capabilities should be used to the maximum to reduce on-site problems.

Many salvage yards also maintain some sizes of both new and used structural shapes in limited quantities.

Railroad rails are usually procured by the railroads in large quantity direct from mills and are stockpiled at division points and other central locations. Both new and re-rail (used) materials may be procured from these sources under emergency conditions. Re-rail from main lines is used by the railroads for the construction of sidings, yards, etc. In some cases re-rail materials are handled by specialty salvage firms to be resold or converted to scrap.

Limited amounts of sheet pile will be found in regional warehouses; however, specialty firms deal in both new and used materials. Sheet pile will be generally found to be more available near the seaboard than elsewhere.

Local sources of structural steel products are easily identified from the yellow pages of telephone directories under the headings of "Steel Distributors and Warehouses," "Steel Fabricators," "Steel Mills," "Steel, Used," etc.

B. Typical Availability

Because of relatively high costs, neither regional nor local sources maintain inventories of all potential sizes and shapes of structural steel products. Large and specialty orders must usually be procured directly on mill order. The stockage will consist of those products and shapes used in relatively large quantities in the area served. These products will generally be limited to those used in the general building work of the area served, usually on medium-sized and smaller buildings, or the equivalent. In a large urban area, however, a fairly wide range of products and sizes would be found among all of the available suppliers. Typical availabilities are indicated in Table E1-7 for "regional warehouses" and for "local suppliers". In all cases, however, availability should be determined before proceeding with design.

Table E1-7

TYPICAL AVAILABILITY OF STRUCTURAL STEEL MATERIALS
AT REGIONAL WAREHOUSES AND LOCAL SUPPLIERS

Material	Local Suppliers	Regional Warehouses
Structural Steel Plate	Usually carry a fairly complete stock of thicknesses from 1/4 in. to 3/4 in. and maybe 1 in. Maximum width of 8 ft, lengths to 20 ft. Any size width and length is easily flame cut. Usually limited to A36 steel.	Will carry same as local suppliers in a larger variety of lengths and widths. Thicknesses up to 4 in. may be available. Equipped for production cutting to size. Other grades available.
Rolled Structural Steel Shapes		
Wide Flange Beams and Columns	Will carry a range of depths in 2 in. increments from about 4 in. up to 16 or 18 in. Weights will be limited to one or two in each size range. Lengths to about 40 ft available in the larger sizes. Any length can be cut. Usually limited to A36 steel.	Carry same as local suppliers plus the larger depths to 36 in. in some quantity. Fairly complete range of weights available in each size range. Some high strength grades (A440, A441, and A242) available in most common sizes. Lengths to 60 ft available in most sizes and weights.
Standard "I" Beams	Generally carry a range of depths from 6 in. to 12 in. to 15 in. Usually only one weight carried. Lengths to about 40 ft available in the larger sizes. Any length can be cut. Usually carry only A36 grade.	All of the local supplies plus the larger sizes to 24 in. and smaller to 3 in. All weights usually available. Lengths to 60 ft. Higher strength grades usually available.

Table E1-7 (concluded)

Light Wide-Flange Columns and Beams; Junior and Miscellaneous Shapes	Fairly large quantities of the smaller sizes, 8 in. depth and less, will be found because of use in light building construction. Weights will be limited to one per size usually. Lengths available to about 30 ft. Usually only A36 grade available.	Will carry a rather complete range of sizes and weights because of heavy use in building construction. Usually not carried in other than A36 grade.
"H" Piles	Usually not carried.	Usually carry complete range of sizes and weights. Longer lengths to 100 ft available in limited quantity. Not carried in other than A36 grade.
Standard Channels	Limited quantity of sizes and weights to about 15 in. carried. Lengths to about 40 ft. Any length can be cut. Usually stocked in A36 grade only.	Carry a fairly complete range of depths and weights to 60 ft length. May have higher strength grades.
Junior Channels	Usually not carried.	Ordinarily carried in all three sizes.
Angles	Fairly complete range of sizes available in the medium and smaller thicknesses.	Will carry same as local supplier plus heavier thicknesses.
Tees	Usually not stocked. Cut on order.	Usually not stocked. Cut on order.
<u>Pipe and Structural Tubing</u>	Usually not carried by local suppliers.	Carry a range of pipe in Standard (Schedule 40) and Extra Strong (Schedule 80) weights. Lengths to 20 ft. Large size (8 to 12 in.) Double Extra Strong possible. Range of square and rectangular tubing is carried.

Dimensioning Practice For Rolled Structural Shapes

Dimensions of rolled structural shapes are generally given in nominal form for depth and width. As a result of a variety of factors including economy in the use of rolls, roll wear, roll re-dressing, etc., a substantial variation of actual dimension from the nominal is produced. The greatest variations result from the process of obtaining heavier weights per foot by "spreading the rolls," thus obtaining greater flange and web thicknesses. This practice permits the rolling of a variety of shapes with the same nominal dimensions on the same set of rolls. Any design or use of structural steel shapes for blast shelter upgrading purposes requires attention to the actual dimensions of the section as opposed to the nominal dimensions. Both nominal and actual (within rolling tolerances) dimensions of the available rolled shapes in the various weights are given in Reference [1], as well as in American Society of Civil Engineers and mill literature. Dimensioning practice varies with the type of shape and is summarized in the following paragraphs.

A. Wide-Flange Beams and Columns (W and HP Shapes)

For a given nominal depth (outide-to-outside of flanges dimension), there is a constant (inside-to-inside) dimension between flanges. The latter remains constant while the former varies with the weight of the section. For example, a W12 section has a constant actual inside flange dimension of about 10-7/8" (actual 10.908") for weights of 40 lb/ft and heavier, while the actual outside depth ranges from 11-7/8" to 14-3/8" with increasing weights per foot. The flange width also varies with the weight, being 8" for a 40 lb/ft section (W12x45) and 12-5/8" for a 190 lb/ft section (W12x190) [1(p.1-17 and 1-38)].

B. Standard "I" Beams (S Shapes)

Differing from wide-flange beam practice, standard "I" Beams are rolled with the actual depth (outside-to-outside of flanges dimension) being equal (within mill tolerances) to the nominal depth. Thus the nominal outside-to-outside depth is the constant, rather than the inside-to-inside depth as in the case of the wide-flange sections. Weight per foot variations result in changes in web thickness and flange width. Thus an 8x4 "I" Beam of 18.4 lb/ft (S8x18.4) has a web thickness of 1/4", a flange (maximum) thickness of 7/16" and a flange width of 4". Increasing the weight to 23 lb/ft (S8x23) results in a web thickness of 7/16" and a flange width of 4-1/8" while the flange thickness remains at 7/16" and the actual depth at 8".

C. <u>Channels (C and MC Shapes)</u>

Standard Channels are rolled and dimensioned in a similar manner to standard "I" Beams with the actual depth (outside-to-outside of flanges dimension) being constant and equal to the nominal depth. For each depth, increased weights result in increased web thickness and flange width while the flange thickness remains constant.

D. <u>Angles (L Shapes)</u>

Variation in weight per foot for Angles is obtained by uniform increase of thickness of legs.

E. <u>Tees (MT, ST, and WT Shapes)</u>

Since Tees are cut from the various beam shapes indicated, dimensioning practice follows that of the basic shape.

REFERENCES

1. Manual of Steel Construction, American Institute of Steel Construction, Inc., Wrigley Building (8th Floor), 400 North Michigan Avenue, Chicago, Illinois 60611; 7th edition, 1970. Note: A later edition is now available.

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Mr. Ken Kaplan
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J. E. Trevor (1)
E. E. Duckstad (1)
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Tacoma, Washington 98411 (2)

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Urbana, Illinois 61801

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Ellis E. Pickering, P.E.
1015 Trinity Drive
Menlo Park, CA 94025

Carl K. Wiehle, P.E.
3311 Fallowfield Drive
Falls Church, VA 22042

UPGRADING BASEMENTS FOR COMBINED NUCLEAR WEAPONS EFFECTS: PREDESIGNED EXPEDIENT OPTIONS II (UNCLASSIFIED)

By: H. L. Murphy*

SRI International (formerly Stanford Research Institute)
Menlo Park, California 94025, July 1980, 272 pages
Contract No. DCPA01-77-C-0227, FEMA Work Unit 1155C (SRI Project 6876)

This report covers the results of the latest phase of a 3-phase project, with the overall objective of developing a set of expedient and engineered techniques, for upgrading the air blast and related effects resistance potential of basements in existing buildings. The purpose of upgrading such basement spaces is to provide shelter when needed by persons in: host areas, where the bulk of the population is expected to be during an attack, that are located at and beyond the 2-psi air blast range, using selected target aiming points and Mt-range bursts; and, risk areas, where shelter is needed that is within 15-minutes travel time of each key worker's place of work and provides potential shelter for 30- to 50-psi air blast ranges, in terms of peak free field overpressure.

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Chapters of this report's main text are devoted to discussions of: background; general principles applicable to upgrading basements; closures for all basement shelter openings/apertures, in terms of principles for providing them; needs to be met in strengthening the structure over shelter candidate basements; some techniques and materials that can be used for such structure strengthening; and, shelters for key workers. In general, the main text of this report is intended for the artisan, the appendices having the more extensive, technical data and discussions.

The titles of the appendices are: Blast-Resistant Design/Analysis General Approach; Plywood Stressed-Skin Panels (Two-Sided Only) as Closures - Design and Fabrication; Plywood Stressed-Skin Panels (Two-Sided) as Beam-Columns; Plywood Use for Closures - Design; Wood Beam and Column Design - Simply Supported; Home Basements Upgrading in Host Areas; Blast-Resistant Design/Analysis of Steel Members; and, Structural Steel Local Availability and Use for Blast Shelter Upgrading.

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* H. L. Murphy Associates, Box 1727, San Mateo, CA 94401 (415-348-2180)
# James E. Beck and Associates, 4216 Los Palos Av., Palo Alto, CA 94306

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The titles of the appendices are: Blast-Resistant Design/Analysis General Approach; Plywood Stressed-Skin Panels (Two-Sided Only) as Closures - Design and Fabrication; Plywood Stressed-Skin Panels (Two-Sided) as Beam-Columns; Plywood Use for Closures - Design; Wood Beam and Column Design - Simply Supported; Home Basements Upgrading in Host Areas; Blast-Resistant Design/Analysis of Steel Members; and, Structural Steel Local Availability and Use for Blast Shelter Upgrading.

The suggestions, guidance and tachnical help of M. A. Pachuta, G. N. Sisson, and D. W. Bensen, FEMA, are gratefully acknowledged, as are the contributions of former colleagues C. K. Wiehle, E. E. Pickering and J. E. Beck.#

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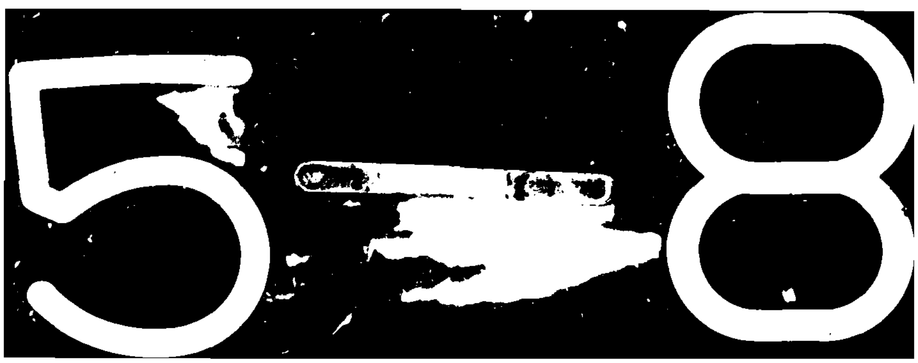
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